URBAN INTERSECTION EVALUATION UTILIZING AVERAGE DELAY PER VEHICLE

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by
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Technical Paper

URBAN INTERSECTION EVALUATION UTILIZING AVERAGE DELAY PER VEHICLE

TO: K. B. Woods, Director
Joint Highway Research Project

February 14, 1962

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

File: 8-4-20
Project: C-36-17T

Attached is a technical paper entitled "Urban Intersection Evaluation Utilizing Average Delay Per Vehicle" by W. W. Scheuer and Harold L. Michael, both formerly or presently members of our staff. This paper was prepared from the Ph.D. thesis prepared by Mr. Scheuer under the direction of Professor H. L. Michael. The complete report has been previously submitted to the Board.

The paper was presented at the last Annual Meeting of the Highway Research Board. It includes the portion of the original report which concerned the development of values for average delay for various volumes of traffic at certain signalized intersections.

The paper is presented for the record and for approval of submission of the paper for possible publication by the Highway Research Board.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:kmc

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Technical Paper

URBAN INTERSECTION EVALUATION UTILIZING AVERAGE DELAY PER VEHICLE

by

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Joint Highway Research Project
File No: 8-4-20
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Purdue University
Lafayette, Indiana

February 14, 1962

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SYNOPSIS

This paper proposes the adoption of "average delay per vehicle" as a criterion to be used to evaluate user satisfaction with intersection traffic conditions, and describes the derivation of certain volume-delay relationships for use as appropriate. The research discussed was performed in connection with the development of a sufficiency rating procedure for urban intersections for the Indiana State Highway Commission.

Theoretical volume-delay relationships are developed for appropriate traffic signal cycle and phase lengths for the case where uniform arrival and fixed-time signals exist or can be assumed.

In order to determine whether or not there was reasonable agreement between the theory and actualtraffic delay, field investigations were undertaken. There were indications of reasonable agreement between the theory and actual traffic performance and that theoretical volume-delay relationships as developed in the paper could be useful as another tool in intersection evaluation.

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INTRODUCTION

It has long been recognized that the at-grade intersection is the major source of trouble and delay for urban traffic; similarly a method has long been desired whereby investigation and planning for the improvement of troublesome locations could be accomplished using logical, engineering procedures.

In the development of an "Intersection Rating" for use in a sufficiency rating study in Indiana, factors which were considered to influence the ability of intersections to serve traffic were divided into two categories, physical factors and traffic factors. The development of a method for evaluating the traffic factors at intersections, however, proved to be a complex problem and resulted in the research reported in this paper.

EVALUATION OF TRAFFIC FACTORS

Logically a physical rating for an intersection should be established on the basis of the visible and/or structural characteristics which affect traffic flow. Correspondingly a traffic rating undertakes the evaluation of "customer satisfaction" with intersection conditions as influenced by interference from traffic control devices, other vehicles and/or pedestrians. These traffic influences are, in the main, variable over wide ranges during short time periods, in contrast to the stability of physical factors.

Average Delay - The Rating Criterion

When a driver passes through an intersection with little or no delay, he is pleased; if he is delayed more than a token amount his ire rises with the length of the delay until a point of frustration and resignation is reached. Beckman, et al, stated it briefly:

"Conditions are good if delay is small; they are bad if delay is large ..... we shall suppose that 'traffic conditions' are fully described by an assessment of the delays that occur" (7)*

"Average-travel-time delay" was selected as a factor descriptive of user satisfaction, and an intersection traffic rating based on average delay was determined to be a goal of this research.

The delay for any given vehicle was defined as the difference between the time at which the vehicle was expected to arrive in the intersection if not interfered with by traffic control devices, other vehicles and/or pedestrians, and the actual time of entry after being subjected to any or all of the above influences. In equation form, this may be stated:

Delay = Actual Entry Time - Expected Entry Time, or briefly:

Delay = Time In - Time Due In.

Traffic ratings based on such an average delay criterion would be applicable to all intersection approaches. The factors affecting delay at intersections, however, are many and variable, depending on type and amount of traffic control, volume, design, etc., and it was not possible to evaluate all pertinent factors under all possible conditions in this study. Traffic delay studies, moreover, have clearly indicated that major delays often occur at signalized at-grade intersections. Consequently, one type of such an intersection was selected for detailed study relative to average delay.

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*Numbers in parenthesis refer to Bibliography items

When a better person thought he discovered his true nature or
he acted, he or she assumed it was a divine power who
had taken hold of him or her during sexual ecstasy. Sometimes
[illegible] [illegible] [illegible]

"Christianity was good to Bailey; it really did. They are [illegible]
to Bailey ..... he [illegible] [illegible] [illegible] [illegible]
[illegible] [illegible] [illegible] [illegible] [illegible]"[1]

"Bailey-Christian-Bailey" was pleased to [illegible]
of some [illegible], and [illegible] [illegible] [illegible]
[illegible] [illegible] [illegible] [illegible] [illegible]

The [illegible] [illegible] [illegible] [illegible] [illegible]
between the [illegible] [illegible] [illegible] [illegible]
[illegible] [illegible] [illegible] [illegible] [illegible]
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Bailey & Antioch Bailey Five - Eighteen Bailey Then - in Bailey:

Bailey - Then [illegible] - Five Ten [illegible]

Bailey [illegible] [illegible] [illegible] [illegible] [illegible]
[illegible] [illegible] [illegible] [illegible] [illegible]
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Fixed-Time Signalized Intersections

To theoretically evaluate average vehicular delay for an intersection approach with a given approach volume and known signal characteristics, certain assumptions were made.

Assumptions

A uniform rate of arrival ofvehicles was assumed. Free-flowing traffic has been reported as following the Poisson distribution by Greenshields, et al, (26) and Matson, Smith and Hurd (52) among others. Nevertheless, because of the sizeable volumes encountered at problem, signalized intersections the average-headway concept seemed a logical and tolerable assumptions, and has been reported by Matson, Smith and Hurd in Traffic Engineering (52). The average headway, often designated as A, is equal to 3600/V, V being an hourly traffic volume. On the average, the first car is assumed to arrive A/2 seconds after the red signal phase begins and this assumption was made for this study. Delayed vehicles were assumed to enter the signalized intersection from the approach at the time intervals reported by Greenshields, et al (26). The first car enters 3.8 seconds after the green begins, the second car enters 3.1 seconds after the first, and successive headways are 2.7, 2.4, 2.2, 2.1 ..... 2.1 seconds, the minimum value of 2.1 seconds being reached at the sixth delayed car.

The amber clearance interval was arbitrarily assigned a length of three seconds, a typical value for urban installations. It was further assumed that if a vehicle were "due in" the intersection in the first two seconds of an amber light, it would enter, being too close to stop conveniently. If a vehicle were "due in" during the final second it was assumed to stop without entering. This is a slightly more

stringent condition than found by Greenshields, who found that "generally the amber signal causes no loss of time" (26).

Below-Capacity Operation

Total Delay for One Red Signal Phase. To obtain average vehicle delay where the approach volume is below intersection capacity, it was convenient to first find a procedure for determining total delay caused by one red phase of length R. This total delay, T, is of course dependent on the frequency of arrival of vehicles. Such a relation is given in Traffic Engineering (52) as (with slight revision):

$$T = nR - \frac{n^2 A}{2} + D_T$$ where

R = red phase length of traffic signal, seconds;

n = number of vehicles delayed in R and during the dispersal of the queue which developes during R and immediately thereafter;

A = average vehicle arrival headway, seconds;

D_T = total theoretical departure delay for n vehicles, seconds, (see Table 1); and

T = total theoretical delay, seconds, all vehicles delayed by one red signal phase.

The total theoretical departure delay, D_T, depends on the number of vehicles delayed and is the sum of the delays to the several delayed vehicles which occur after the red light changes to green and is dependent on the starting characteristics of the entire queue of vehicles. Using the entry times as found by Greenshields, et al, Table 1 was prepared. The value of D_T when n has the value of 1 to 32 are given in this Table.

Use of the above relationship for T assumes a green interval sufficiently long to move through the intersection all those vehicles stopped as well as those arriving before the queue disperses. Use of the

relationship also requires knowledge of the value of n, the number of cars delayed. This may be obtained from the relation $n = \frac{R + 4.75}{A - 2.1}$ where n, R and A are as previously defined (52) and fractional values of n are rounded to the nearest integer.

An example of the use of the preceding formulae to obtain total delay for one red signal phase follows:

Given : A street with one approach lane from a certain direction.

$V = 300$ vehicles (traffic volume in vehicles per hour).

$R = 30$ seconds (red phase length).

Find: Total Delay, T, for one red signal phase assuming all delayed vehicles are dispersed during following green phase.

Solution:

$$A = \frac{3600}{V} = \frac{3600}{300} = 12.0 \text{ seconds}$$

$$n = \frac{R + 4.75}{A - 2.1} = \frac{30 + 4.75}{12.0 - 2.1} = \frac{34.75}{9.9} = 3.51, \text{ use } 4$$

$D_T = 32.3$ seconds (from Table 1)

$$T = nR - \frac{n^2 A}{2} + D_T = 4(30) - \frac{16(12)}{2} + 32.3 = \underline{56.3} \text{ seconds}$$

Through use of the method just discussed, total delay due to one red signal phase was calculated for different red phase lengths and different approach volumes for a single approach lane. Results are shown in Figure 1. No values are shown for volumes below 200 vehicles per lane per hour because it was felt that the uniform-arrival assumption becomes untenable at such low volume and that a traffic signal usually would not be warranted at such low volumes. Two hundred vehicles per lane per hour, therefore, was arbitrarily selected as the lower cut-off volume.

Average Delay per Vehicle. Total delay per red phase is the same as total delay per signal cycle, if conditions are such that the backlog can be eliminated during the green phase. Conversion to theoretical average delay per vehicle, D, is then simply a matter of multiplying total delay per cycle by the number of cycles per hour, and dividing by the hourly volume:

$$D = T\left(\frac{3600}{L}\right) \quad \left(\frac{1}{V}\right)$$

where

D = theoretical average delay per vehicle, seconds;

L = total cycle length, seconds, and

T and V are as before

It will be recognized that 3600/V is equal to A, the average headway, and the expression may be simplified to $D = TA/L$.

The calculation of theoretical average delay per vehicle using the methods just discussed can be performed for any cycle length with any red phase length for an approach lane to an intersection. The formulae, however, do not include delays due to pedestrians, trucks, turning movements, etc., and are only applicable when all vehicles which are delayed by the red phase of the signal are able to clear the intersection during the following entry time.

The maximum number of vehicles which can pass through an intersection in accordance with the assumptions made in an hour is here called the capacity of the intersection. The calculation of this capacity, or maximum value to which the formulae used in this section apply, was the next step in the delay research reported here.

Approach Capacity

Calculation of Capacity Volumes. Using the assumptions previously

mentioned, the capacity of the approach lane may be calculated if the signal characteristics are known.

The first step is to find the maximum number of vehicles that a specified green phase will permit to enter the intersection, entry time, E, being equal to cycle length, L, less non-entry time, R+1,: E = L - (R+1). Non-entry time is R+1 because of the assumption that entry is permitted during the entire cycle except for the red phase "R" and the final second of the amber clearance interval. After determination of the entry time for a particular cycle length and red phase length, the maximum number of cars delayed by the red phase but which can clear the intersection in the following entry time, can be determined from Table 1. The maximum number of cars is that n whose departure delay (3rd column of Table 1) is equal to or just less than the available entry time. This n is also the n in the relation $n = \frac{R + 4.75}{A - 2.1}$ previously utilized. With n and R known, A may be readily found; then 3600/A = V, the capacity in passenger cars per lane per hour.

Kind of Capacity Calculated. This method of calculating capacity is based on the assumption of uniform arrival of vehicles and as the volume on the lane approaches this capacity some vehicles, because of unequally spaced arrivals, will be delayed more than one red phase. Such a condition when it becomes serious would be operation at above practical capacity as customarily defined.

On the other hand, it is recognized that there is such a thing as the "pressurized intersection" (10, 77). Where congestion exists, drivers tend to be more tense and alert and react more quickly, permitting higher volume flow than is indicated by using normal headway values. In such cases, possible capacity volumes would be higher than those calculated by the method herein presented.

It thus appears that capacities as calculated above are "intermediate capacity" values, directly analogous to values taken from the Highway Capacity Manual capacity curves (18, 68) and should be subject to approximately the same adjustments to get either practical or possible capacity.

Comparison with Other Capacity Values. The question, however, may be raised as to how well capacity values calculated by the method just outlined agree with values using the accepted methods outlined in the Highway Capacity Manual, the Manual of Traffic Engineering Studies, and/or Circular 376 of the Highway Research Boards Research Correlation Service (18, 57, 68).

To compare the methods it was necessary to establish a common basis. Investigation of Greenshields' work (26) indicated his departure headways, which are the basis of the capacity method used here, to have been based on the following conditions:

1. Passenger vehicles only - no truck or bus effect;
2. No turn effects - hence no pedestrian effect;
3. Parking permitted on each approach; and
4. Two-way traffic.

A very common condition existing on urban streets is two-way traffic with one lane carrying traffic in each direction and with parking permitted. For this research, therefore, a standard lane was defined as one which was ten feet wide, with parking adjacent to it, with an adjacent lane carrying traffic in the opposite direction and located in the fringe of the business district. For these conditions and in the range of typical street approach widths of twenty to forty feet, the

Highway Capacity Manual and Circular 376 (18, 68) give the average effect of parking as equivalent to twelve feet of approach width.

The same Capacity Manual and the charts of Circular 376 also indicate that the capacity for a twenty-two foot approach (one standard ten-foot urban traffic lane plus twelve feet for parking) in the fringe area with parking is 1250 vehicles per hour of green. This value, however, includes the effect of ten percent commercial vehicles and ten percent each of left and right turns. For the condition of no commercial vehicles and no left or right turns (the condition for which the method used in this study was discussed) an adjustment of plus 25 percent is necessary, resulting in an adjusted capacity of 1560 cars per hour of green using Capacity Manual methods. This capacity is also defined by the Capacity Manual as between possible and practical capacity.

To compare the methods, a fifty-second cycle was selected, with red phases of twenty, twenty-five and thirty seconds. The Capacity Manual cautions against using amber time as entry time, so the corresponding entry times for the Capacity Manual method were twenty-seven, twenty-two and seventeen seconds. The following table shows the capacity comparisons:

Comparison of Calculated Capacities for 50 Second Cycle Length

	Capacity Manual Method		Departure Delay Method	
Red Phase	Entry Time	Calculated Capacity	Entry Time	Capacity
20	27	844	29	867
25	22	688	24	695
30	17	531	19	524

Thus, when the Manual capacity of 1560 cars per green hour was converted to cars per clock hour by multiplying by the time factor, resulting capacities agreed closely with those computed by the method

based on departure delays. It will also be noted that the latter method reflects the added efficiency of long green phases and vice versa. Indicated capacities are higher than those obtained by Manual methods where greens are long, and lower where green phases are short.

This comparison indicates that the "intermediate capacity" for one ten-foot lane carrying only passenger cars inhibited only by each other and signal characteristics, is about 1560 cars per hour of green. Recent investigations indicate that the capacity of a lane in urban areas is near this value. George Sagi reports a theoretical value of 1714 vehicles (cars) per lane per hour for the situation where "conditions are ideal and when the steady flow of vehicles is periodically interrupted by stops and where the go time is also limited" (66). Sagi's value of 1714 (possible capacity), divided by 110 percent to reduce it to "intermediate capacity" gives a value of 1558 cars per green hour, which compares closely to the value previously determined.

The Standard Lane Concept. The 1560 capacity value, however, considers only passenger car interaction and signal interference, and is referred to in this study as the standard lane capacity; unfortunately situations with such standard lanes are not common, and realities must be faced.

The most satisfactory method presently available for evaluating the many disruptive influences found at the typical intersection is the Manual procedure; calculated capacities then include the effects of all factors for which effects have been thus far evaluated.

If the intersection-approach capacity per hour of green as calculated by Manual methods (and including the effects of all factors) is divided by 1560 (the standard lane capacity) a value is obtained which may be called the "effective number of lanes" of traffic which may flow. If, then, an actual hourly traffic volume is divided by the "effective

number of lanes", the result is that part of the total flow which may be considered to flow during that period in one standard ten-foot approach lane. Such values are called standard lane volumes in this report.

It was necessary to establish a common volume measure such as the standard lane volume (which is in terms of cars per hour), because the departure data previously used are valid only for the case where the traffic consists of nothing but passenger cars. Determination of the standard lane volume for each intersection approach may be expedited by graphical means such as the N-chart of Figure 2.

Above-Capacity Operation

Above-capacity operation is, by definition, that case where more vehicles arrive during a signal cycle than may enter the intersection. As long as the volume remains constant, the backlog of vehicles increases with every signal cycle. There is then no singal value for average delay as in below-capacity operation; average delay now depends, among other things, on how long the overload has been in existence.

Adoption of the Five-Minute Delay Period. It clearly follows that if a measure of average delay were to be established, it had to be applied on an arbitrary basis; for this prupose a five minute period was selected, and average delay was calculated for the first five minutes of overload. A five minute period was selected because of sustained above-capacity volume is usually of such short-term duration.

For vehicles which arrived, were delayed, then entered the intersection during the allotted five minutes, delay was figured as time in less time due in. If a vehicle had not entered the intersection at the end of the five-minute period but could have if there had been no delay, its delay was taken as 300 seconds minus time due in. The actual

delay for these vehicles was hence understated and the average delay to all vehicles over the five minute period was slightly less than if these vehicles had been ignored altogether.

Electronic Computer Utilized. Having previosuly established reasonable cycle lengths, red phase lengths and the approach capacity for reasonable combinations of signal cycle and red phase lengths, it was not difficult to set up a computer program for the above-capacity phase of the study. Some such computational method was a virtual necessity because numerous points were required to fix each of the many curves accurately. Figure 3 shows the flow diagram used in setting up the problem for solution on Purdue University's Datatron.

Average Delay Charts

Graphs showing the relationship of theoretical average delay D to standard lane volume were prepared and are shown in Figures 4 through 12. This relationship was calculated for cycle lengths of from 40 to 80 seconds in 5-second increments, thus covering the common cycle lengths, and for various red phase lengths in 5-second increments over a range considered appropriate to each cycle length.

The portions of the graphs reflecting below-capacity operation begin at a standard lane volume of 200 cars per hour and show relatively little increase in theoretical average delay with increasing volume to the points of discontinuity. These points represent capacity operation. Theoretical average delays for above-capacity operation are shown by the portions of the curves to the right of the points of discontinuity and indicate rapid increased in average delay for this condition. The above capacity curve portions were terminated when the theoretical average delay approached sixty seconds per vehicle.

Measured Delay at Fixed-Time Signalized Intersections

Verification of Theoretical Relations

"Logic is an organized way of going wrong with confidence" is a quotation attributed to Charles Kettering (45). To guard against such an occurence, it was considered necessary to investigate a number of existing signalized intersections to determine if, in fact, actual traffic behavior compared with the theoretical volume-delay relationships.

Choice of Intersections to be Studied

Approaches to be studied were selected with several criteria in mind. First, it was necessary for the approach to be long enough that delay might be accurately measured. This frequently caused a side street to be included in the zone length, with attendant complications. Second, the approach traffic had to flow at relatively high volume, at least during some parts of the test period. Overloads were desired so appropriate data for above-capacity conditions could be obtained. Actual overloads for as long as five minutes, however, were found to be few and resulted in the obtaining of less data for the overload condition than was desirable.

It was also desired to keep the approaches as simple and uncorrupted as possible. Locations chosen, with appropriate features noted, are shown in Figures 13 through 16.

All measurement zones were approximately level, and although two were near the crests of long hills, the beginnings of the zones were sufficiently far from the hill crests that no difficulty was experienced from delayed heavy trucks.

Care was also taken to choose approaches where the vehicles would arrive without regimentation from preceding signals.

Procedure

Average-delay data were measured in the field using two stop watches (started simultaneously so that both read the same at any instant of time) and an Electromatic radar speed meter. Time of vehicle passage at Station "A" was observed, as was the radar speed. With zone length known, "time due in" was simply computed. "Time in" was observed at Station "B", using the second stop watch. At both stations the last two license plate characters (letters and/or figures) were recorded for identification and data matching purposes.

Analysis of the Data

Technically-good data were obtained for 479 five-minute periods at the four intersections. During these time periods 16,472 vehicles passed the "A" Stations and 13,437 of these were satisfactorily matched with "B" Station data, resulting in 318,751 seconds (about 88.5 hours) of delay.

Early in the calculations, it became apparent certain restrictions were needed. In some cases, negative delays (the vehicle arrived at Station "B" before his calculated time due in) were obtained. It was concluded that such delays of small magnitude were tolerable and attributable to slight errors in reading time and/or speed. At low speeds in particular, a one-mile-per-hour error affected the transit time by two or three seconds. The tolerable limit for negative delays was set at minus three seconds; values below that level resulted in rejection of that data for error of underterminable cause. Negative values within the set limit were considered to result from random error and were figured to average out in the long run.

To eliminate another error source, it was necessary to insure that the radar speed measured represented the drivers freely-selected,

uninhibited speed. In some cases, drivers for some reason slowed before passing "A" Station. After some consideration, eleven miles per hour was set as the minimum speed that would qualify as uninhibited; any lesser recorded radar speed resulted in elimination of that vehicle.

Vehicles passing these requirements, and their delays, were totalled, and average delay was calculated. If delays were obtained for less than fifty percent of the cars passing "A" Station in any five-minute period, the period was dropped from consideration.

"A" Station volume for the five minutes times twelve was considered the approach volume, stated in hourly terms as is customary. This volume was then converted to standard lane volume by multiplying the ratio 1560 divided by the capacity calculated by Manual methods.

Northwestern Avenue, by virtue of its long no-parking zone qualified for capacity calculation at the intersection as a street with parking prohibited. Field observation, however, indicated that drivers did not utilize the extra lane for movement purposes, except when delays were very high and "the pressure was really on".

Figures 17 through 23 show the relation of the measured field delay data (individual car delays averaged over 5-minute intervals) to the appropriate theoretical curves. Figures 19A and 20A show the relationships considering Northwestern Avenue as an approach with no parking; agreement was poor. With Northwestern Avenue capacity calculated in accordance with the way the approach was actually used, i.e. as if parking was to the intersection, averages of measured delays were scattered around the curves as shown in Figures 19B and 20B.

Overall agreement of the five-minute averages with the theoretical curves was considered to indicate that the actual average delays to motorists at such intersections are similar to those which could be obtained from the theoretical curves. The variability in the average delays found in the field for five-minute periods is not ususual when one considers the tremendous variability one finds in drivers and in their movement in an automobile through an intersection. The points, however, are well scattered around the theoretical curves and if one obtains an average delay value for each increment of volume, say 50 cars, the points plot reasonably close to the theoretical curves. The field data shown in Figures 17 through 23 (except 19A and 20A) for the below-capacity conditions are almost equally located above and below the theoretical curves. The data in Figures 19A and 20A are not located around the curve because, as discussed earlier, traffic did not use this location as a two-lane approach as it could have but as a one-lane approach. When plotted with the capacity of a one-lane approach as in Figures 19B and 20B the points become scattered around the theoretical curves.

As also mentioned earlier, it was not possible to obtain the desirable amount of data for the above capacity condition. This was particularly true of the intersections shown in Figures 17, 18, and 21. Although additional data would have been desirable there is a reasonable scatter of the five-minute averages around the theoretical curves in Figures 19B, 20B, 22 and 23. In Figure 19B, for example, the average of all delay values for each increment of 50 for volumes of 450 to 600 (the above capacity volumes) are almost on the theoretical curve (see Figure 19B). Similar results occurred for the intersections shown in

Figures 20B, 22 and 23 and provided an indication that the overall average delay for many motorists would be similar to that which is indicated by the theoretical curves.

As a result of the field verification study, the following conclusions were drawn.

First, it was concluded that for the appropriate standard lane volume, (i.e. one based on valid capacity calculations) the theoretical curves developed in this study give reasonable estimates of the average vehicular delays to be expected under appropriate signal conditions.

Second, care must be taken to calculate approach capacity on a basis conforming to the way drivers use the approach.

Third, more refined and precise methods of capacity calculation would be most valuable, particularly at intersections with rather unusual characteristics such as separate turning lanes and/or separate signal indications.

Fourth, additional field data for the above capacity situation would be very desirable.

Summary

A general criterion has been established whereby the delay characteristics of each approach to an intersection provide a basis for approach evaluation. Delay characteristics reflect the ability of an intersection to handle the required traffic movement and offer the possibility of factually evaluating intersection efficiency.

Specific theoretical delay data are developed for those intersections where control is by fixed-time signals and where uniform arrival

may be assumed. Field investigations provided an indication that average vehicular delays which occur at intersections controlled by isolated fixed time signals can be estimated satisfactorily by the solution of the mathematical models employed. These models included consideration of intersection capacity, approach volume and distribution, intersection control characteristics and vehicular spacing when entering the intersection.

The procedure used for determining delays is applicable to all intersections, regardless of type of control, and should be a useful tool in comparing approach efficiency, evaluating control changes, or in evaluating the efficiency of urban intersections.

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TABLE 1

Total Departure Delay, D_T

Car No. "n"	Departure Headway, D_n	Departure Delay to nth Car $\leq D_n$	Total Departure Delay n Cars, D_T
1	3.8	3.8	3.8
2	3.1	6.9	10.7
3	2.7	9.6	20.3
4	2.4	12.0	32.3
5	2.2	14.2	46.5
6	2.1	16.3	62.8
7	2.1	18.4	81.2
8	2.1	20.5	101.7
9	2.1	22.6	124.3
10	2.1	24.7	149.0
11	2.1	26.8	175.8
12	2.1	28.9	204.7
13	2.1	31.0	235.7
14	2.1	33.1	268.8
15	2.1	35.2	304.0
16	2.1	37.3	341.3
17	2.1	39.4	380.7
18	2.1	41.5	422.2
19	2.1	43.6	465.8
20	2.1	45.7	511.5
21	2.1	47.8	559.3
22	2.1	49.9	609.2
23	2.1	52.0	661.2
24	2.1	54.1	715.3
25	2.1	56.2	771.5
26	2.1	58.3	829.8
27	2.1	60.4	890.2
28	2.1	62.5	952.7
29	2.1	64.6	1017.3
30	2.1	66.7	1084.0
31	2.1	68.8	1152.8
32	2.1	70.9	1223.7

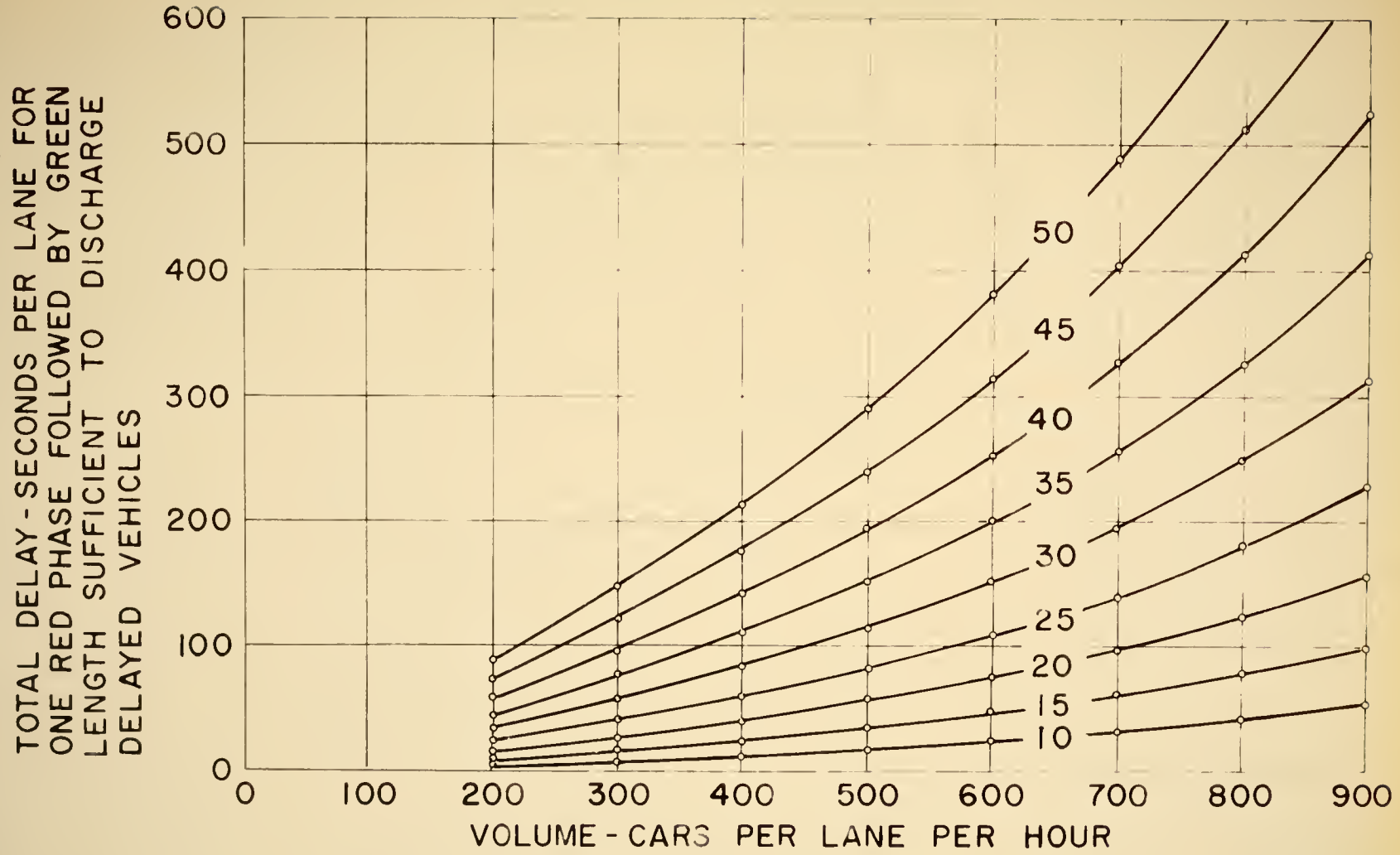


FIGURE 1. RELATION OF TOTAL CYCLE DELAY TO VOLUME FOR VARIOUS RED PHASE LENGTHS.

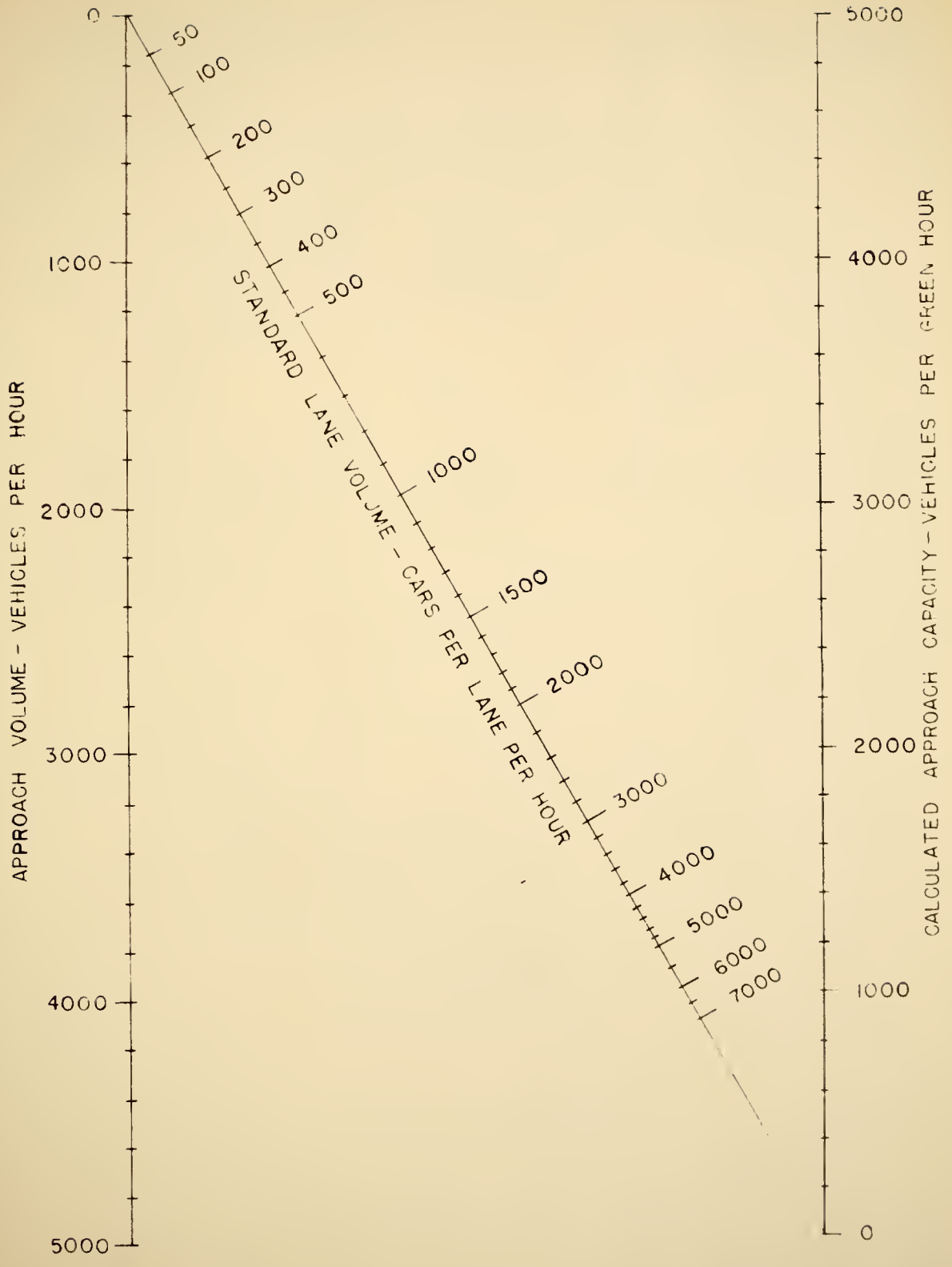


FIGURE 2. DETERMINATION OF STANDARD LANE VOLUME.

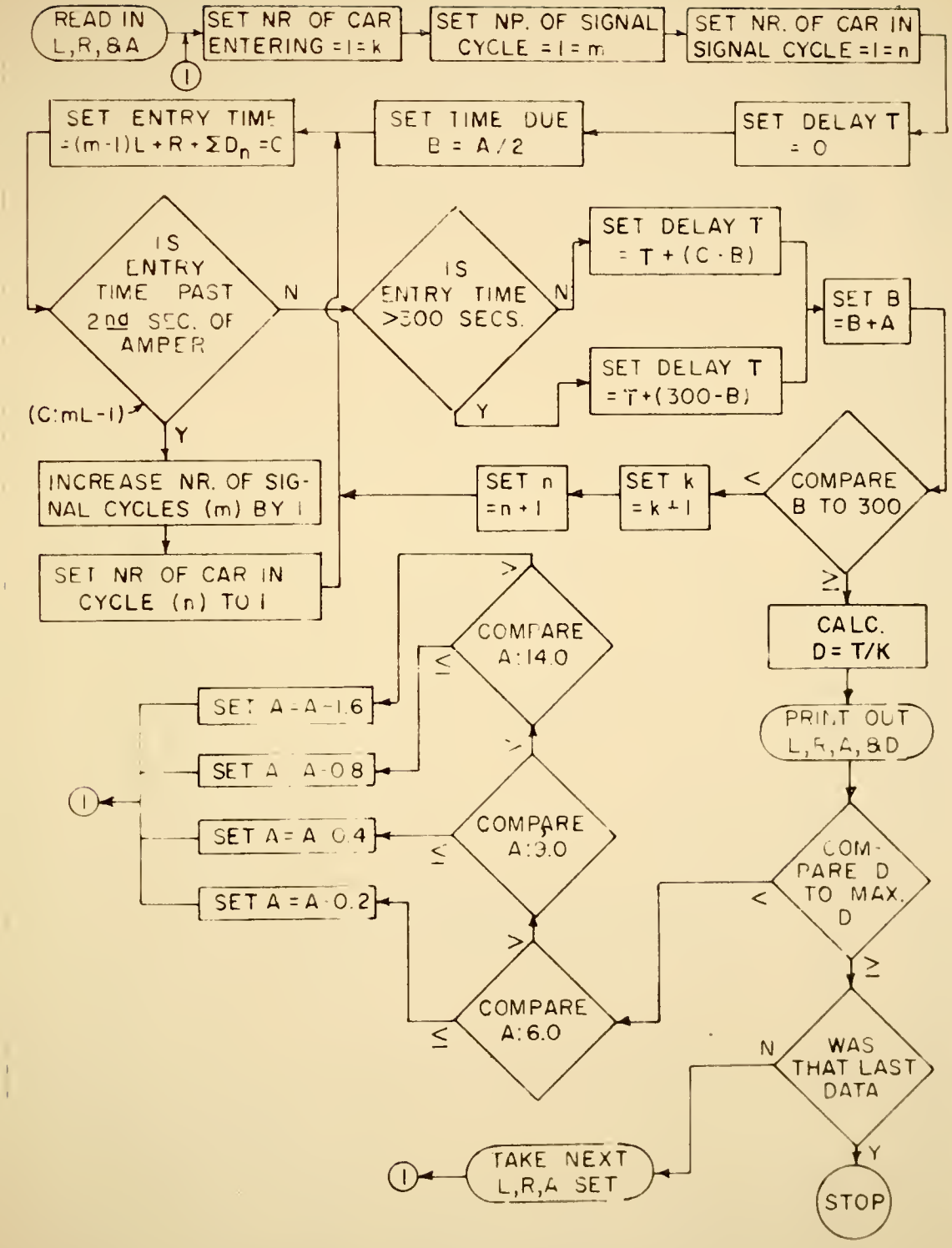


FIGURE 3. FLOW DIAGRAM FOR ELECTRONIC COMPUTATION OF AVERAGE DELAY AT ABOVE-CAPACITY APPROACH VOLUMES.

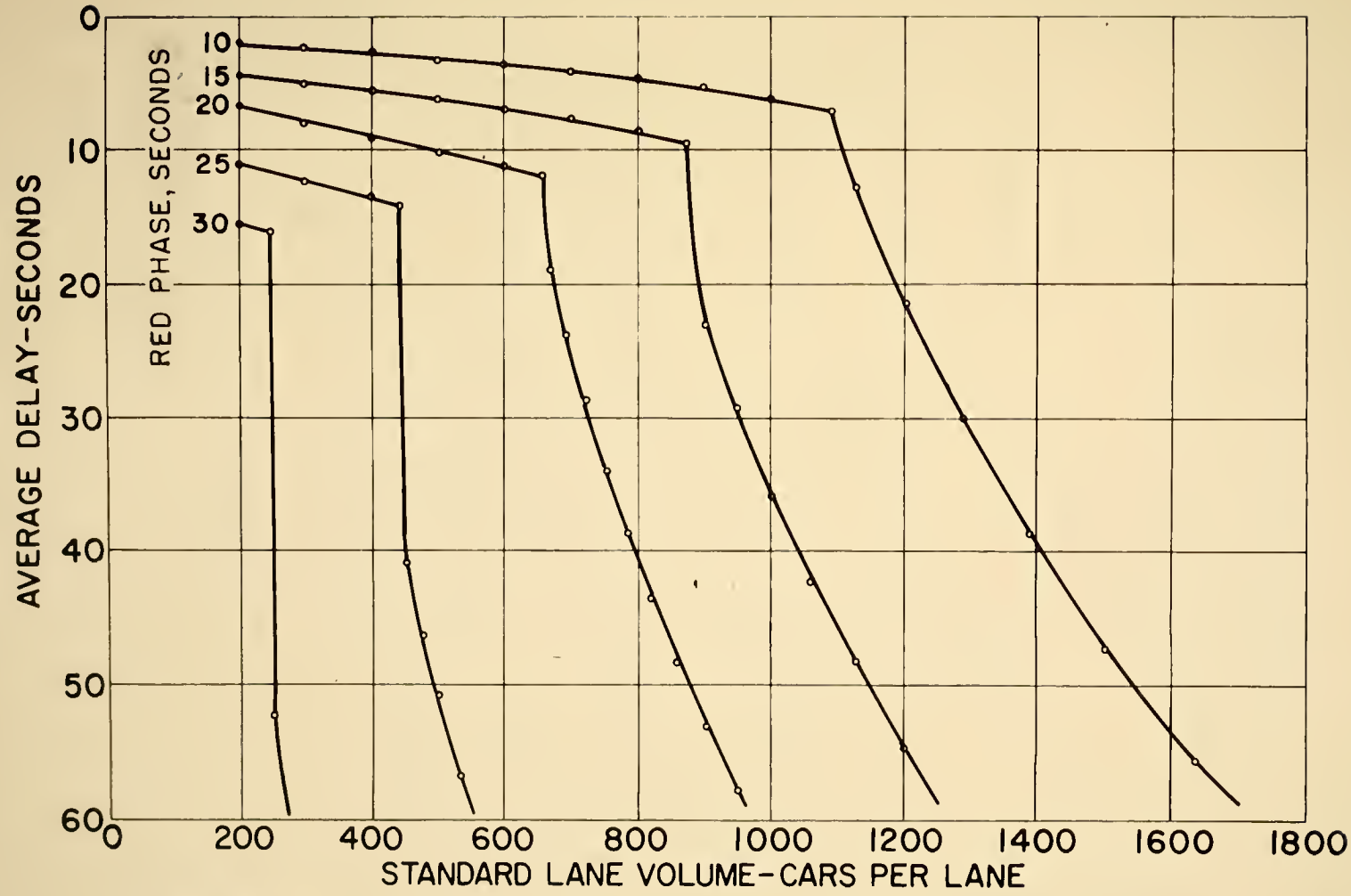


FIGURE 4. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 40 SECOND CYCLE.

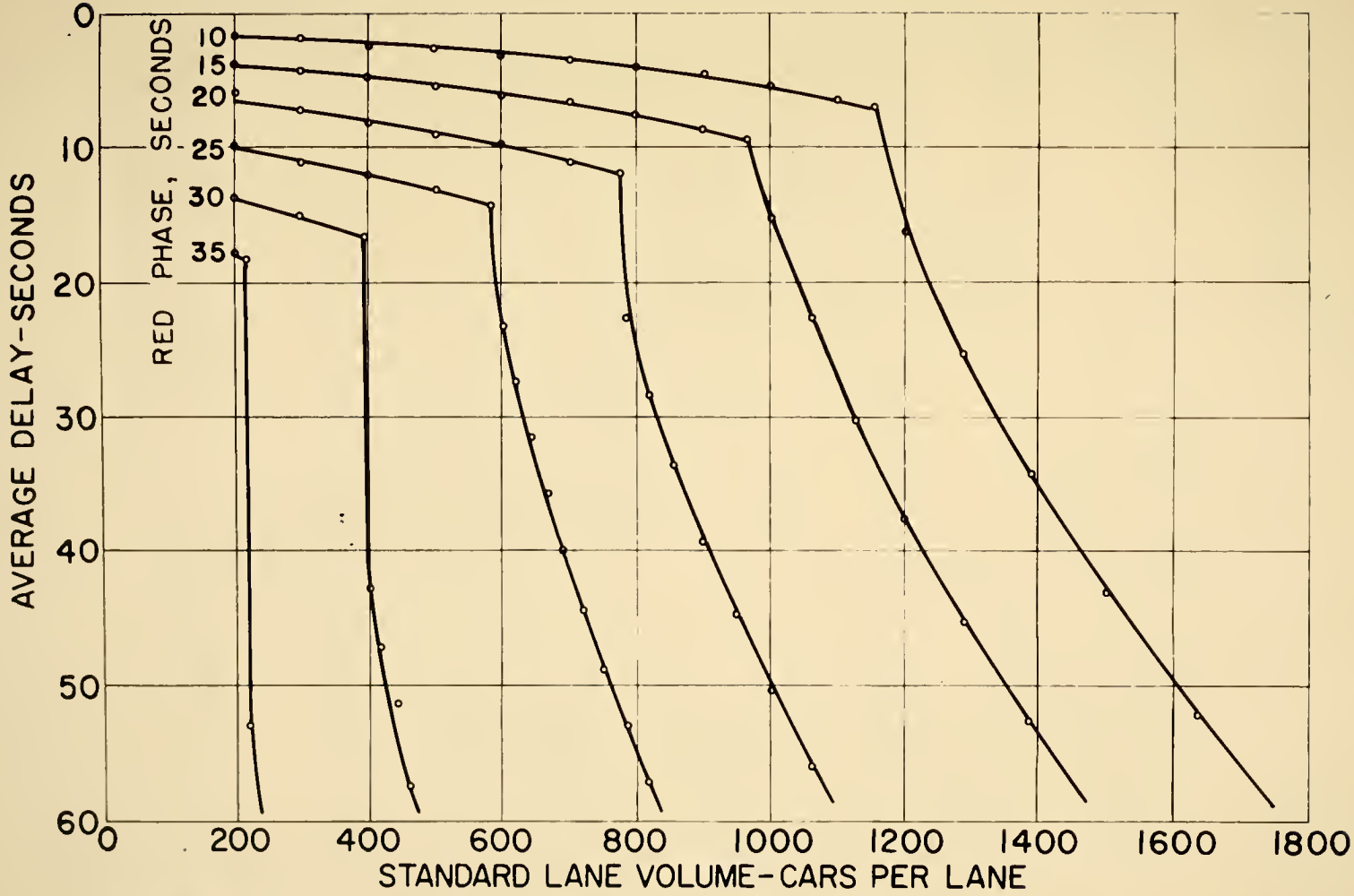


FIGURE 5. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 45 SECOND CYCLE.

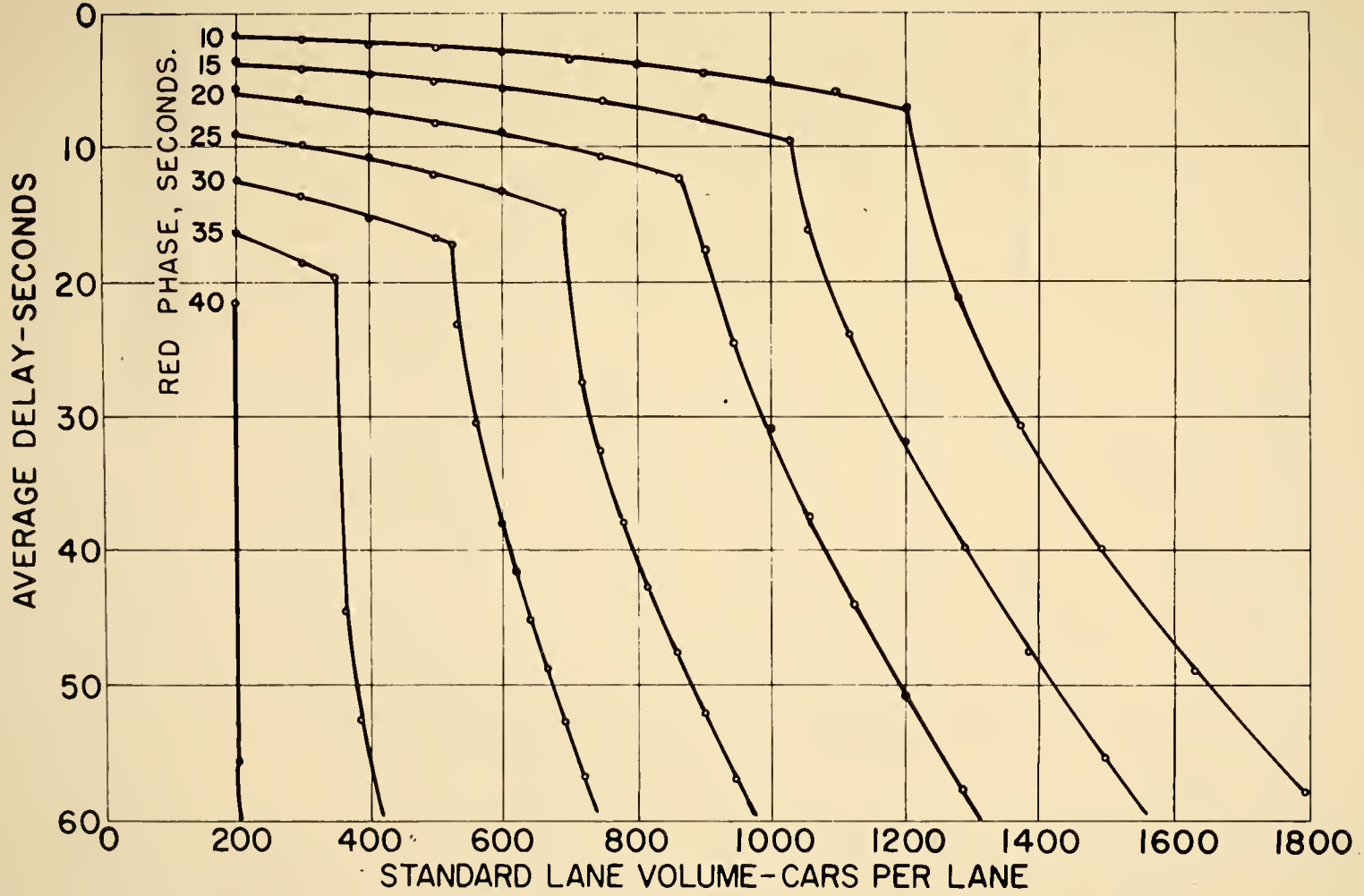


FIGURE 6. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 50 SECOND CYCLE.

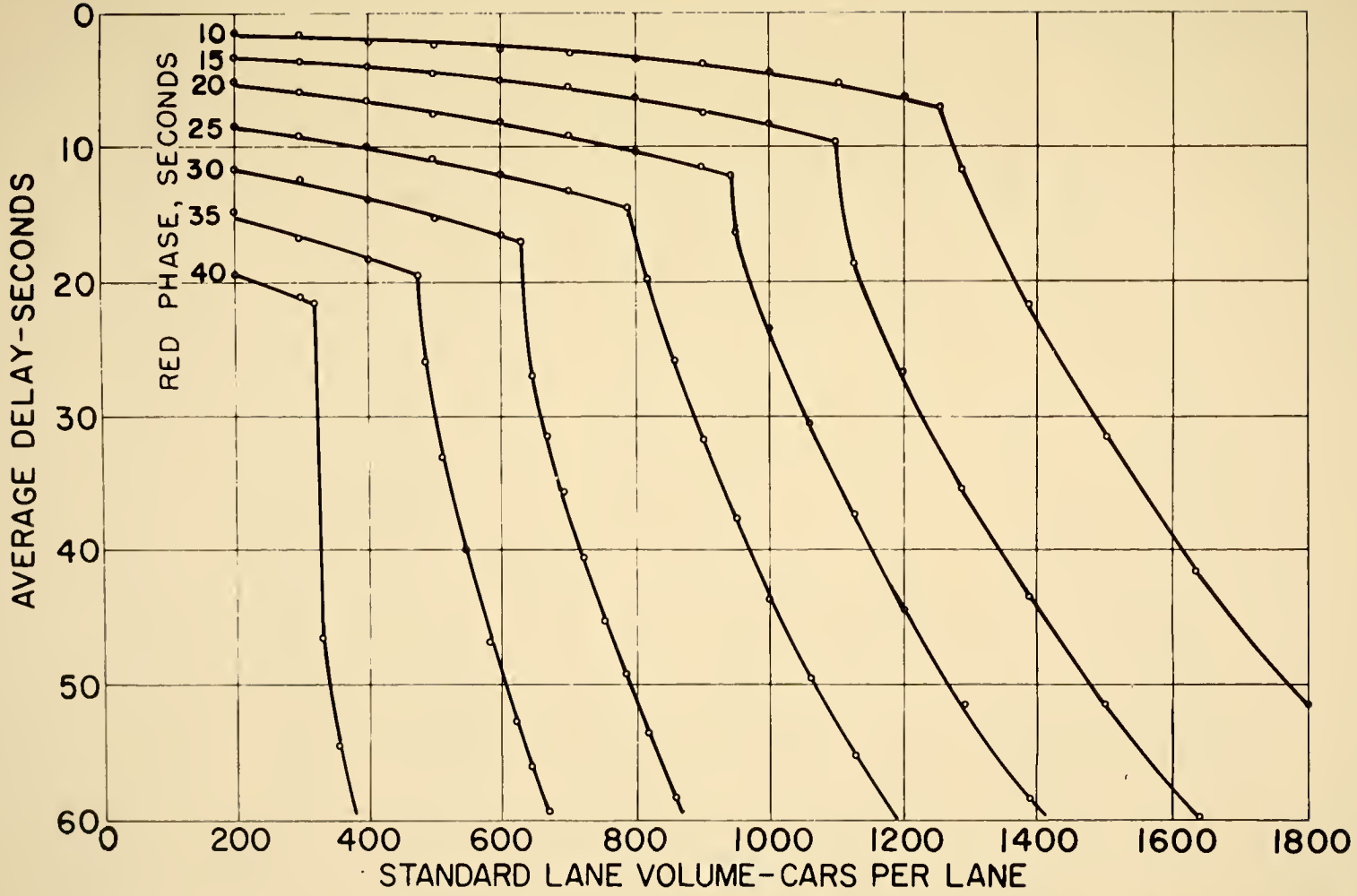


FIGURE 7. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 55 SECOND CYCLE.

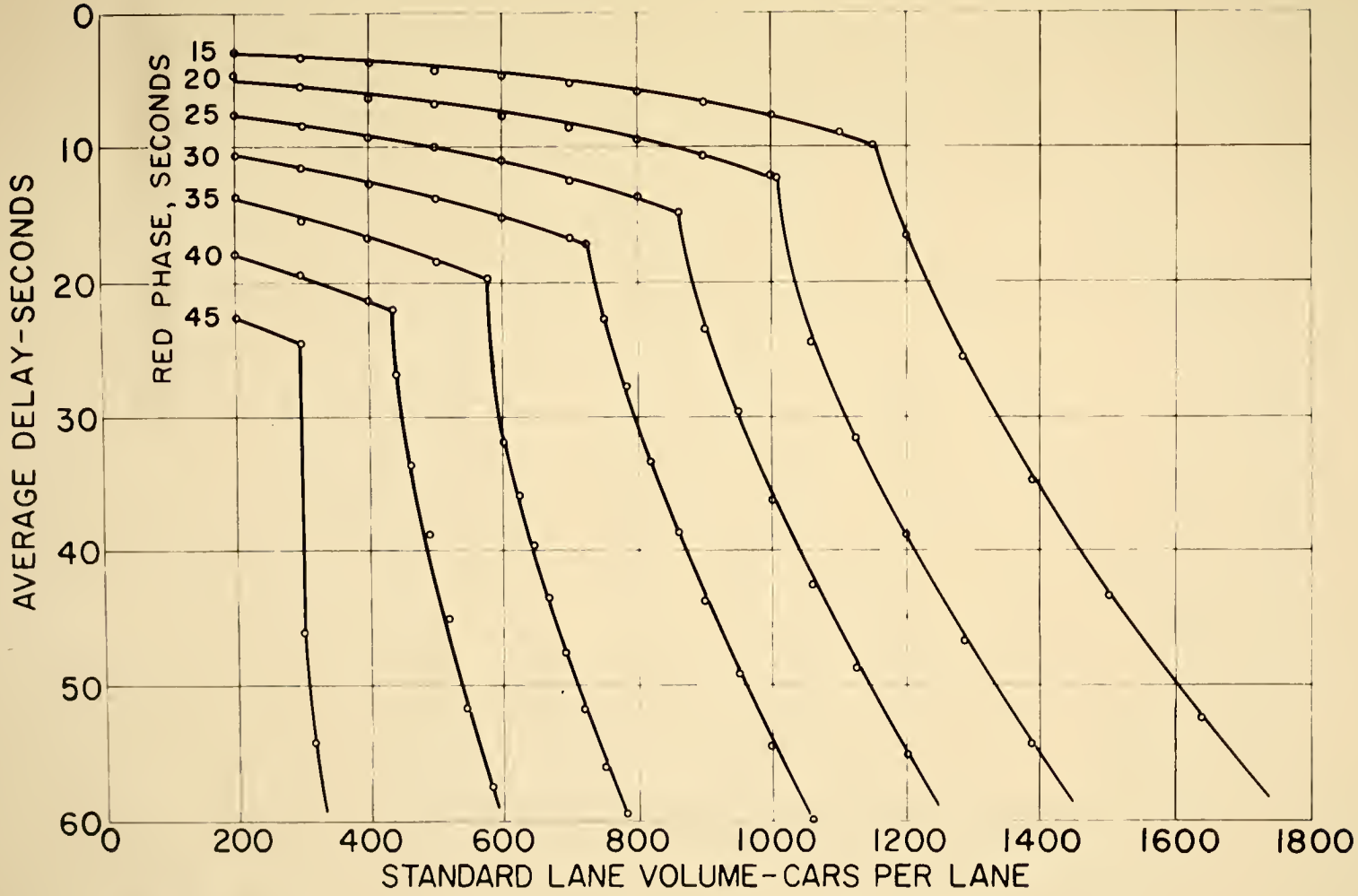


FIGURE 8. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 60 SECOND CYCLE.

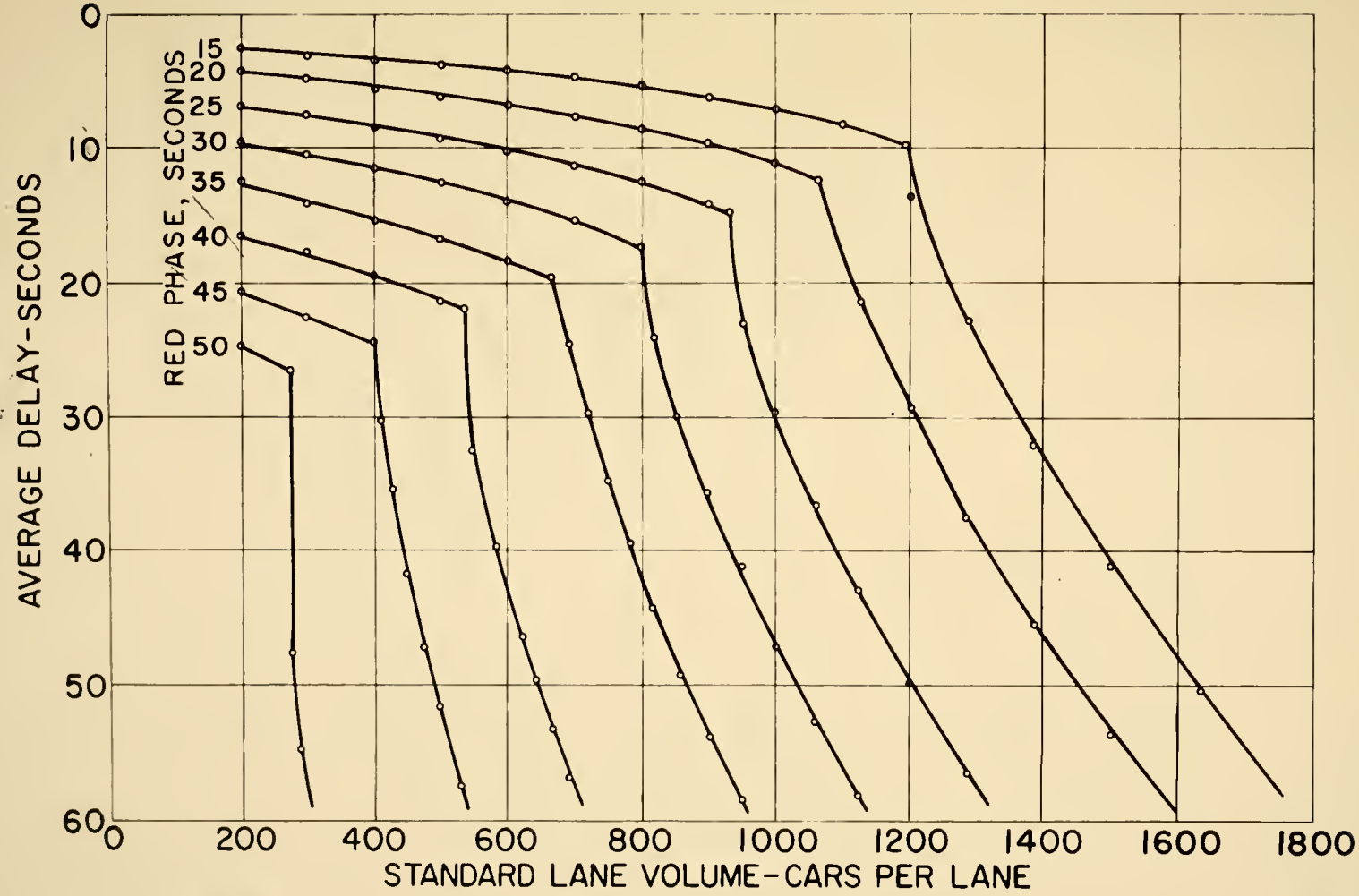


FIGURE 9. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 65 SECOND CYCLE.

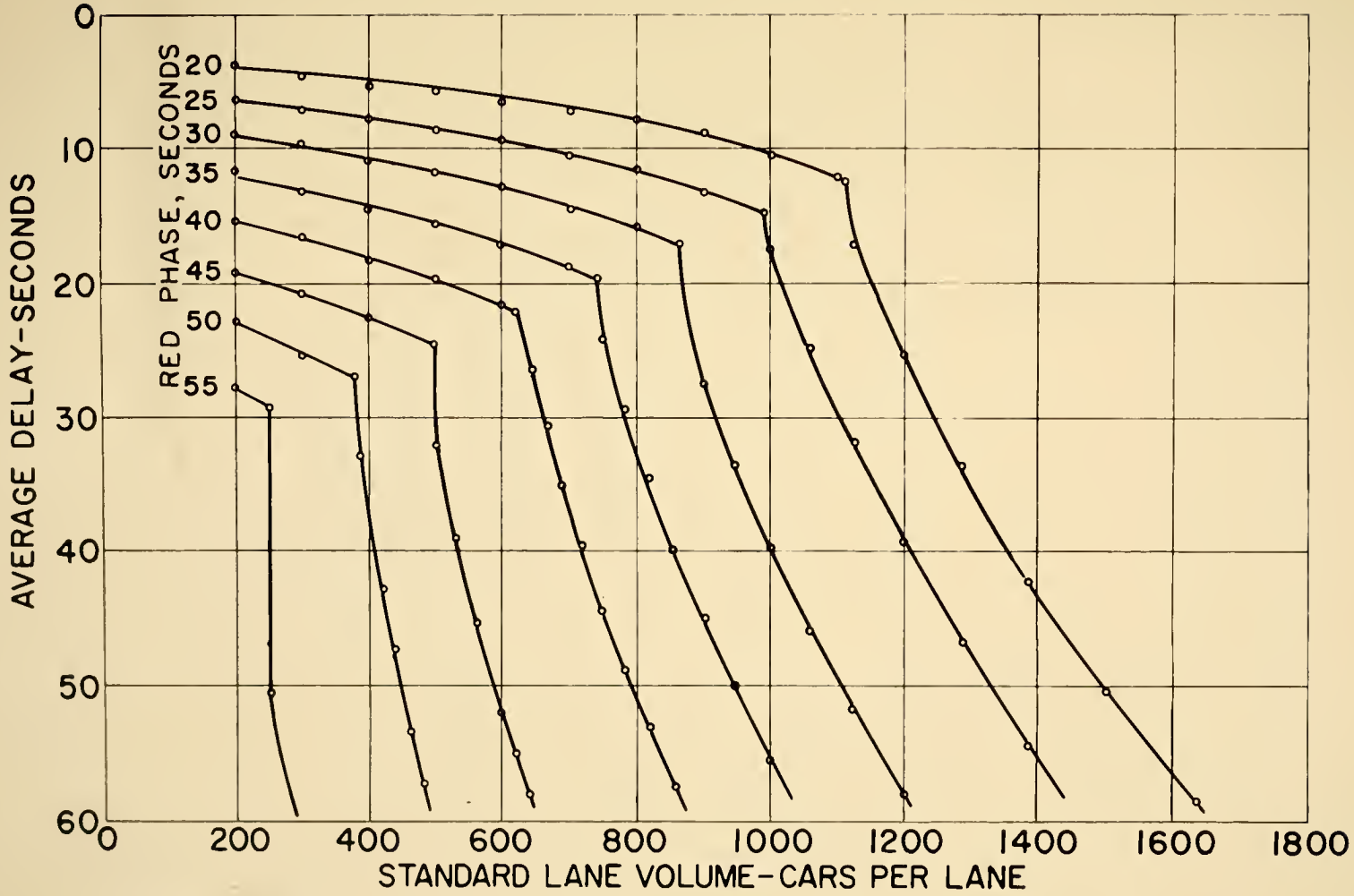


FIGURE 10. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 70 SECOND CYCLE.

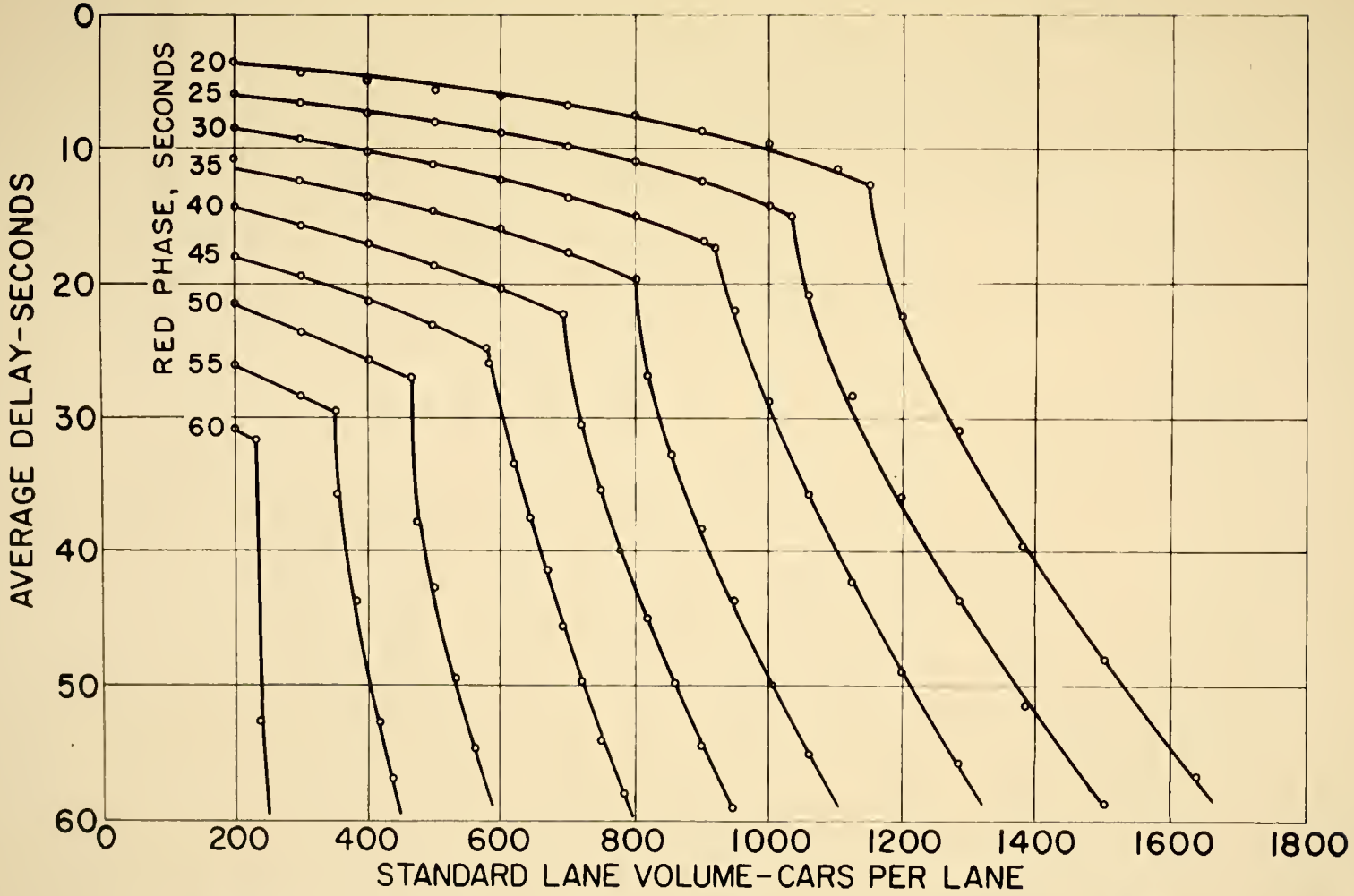


FIGURE 11. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 75 SECOND CYCLE.

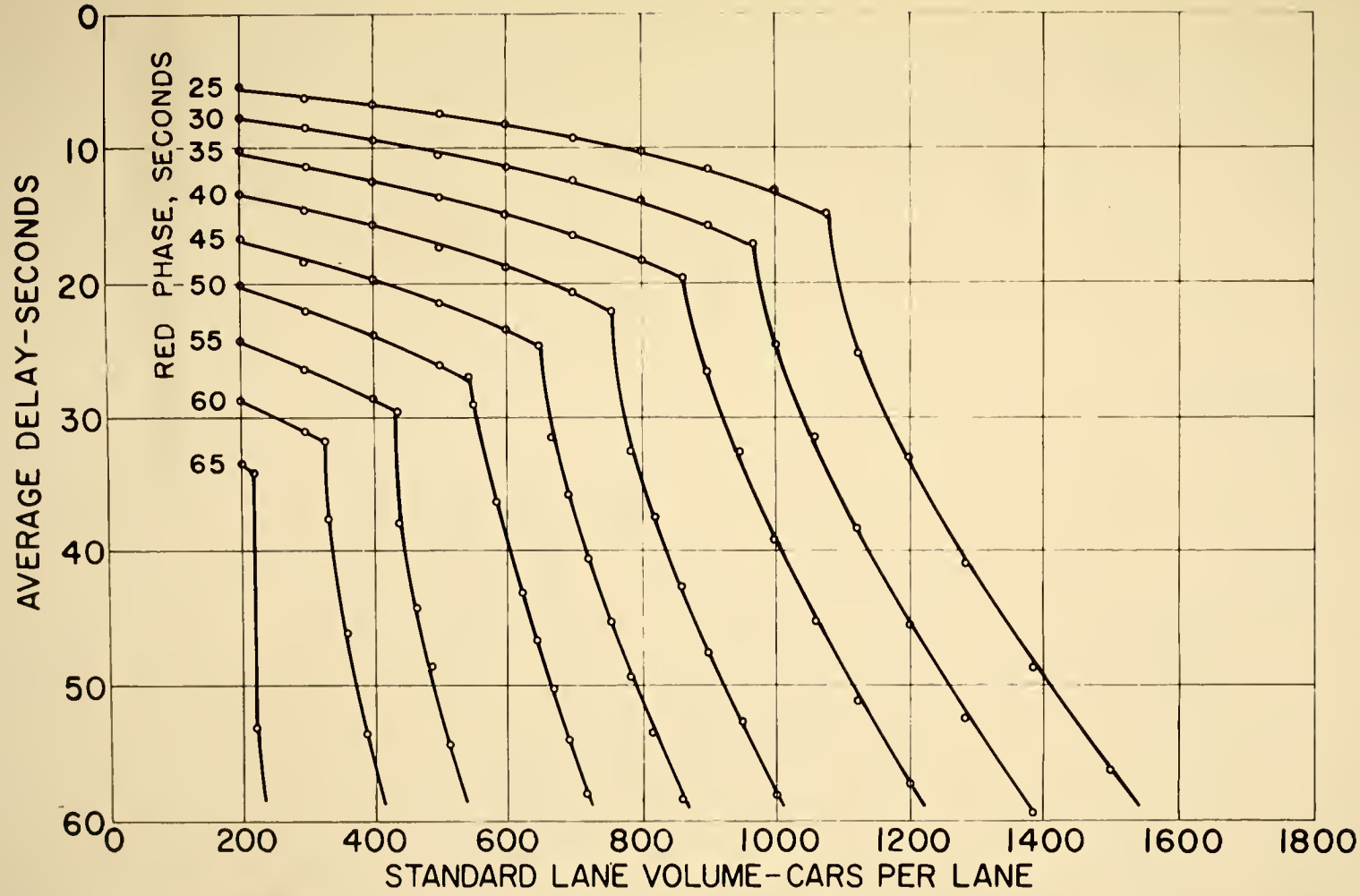
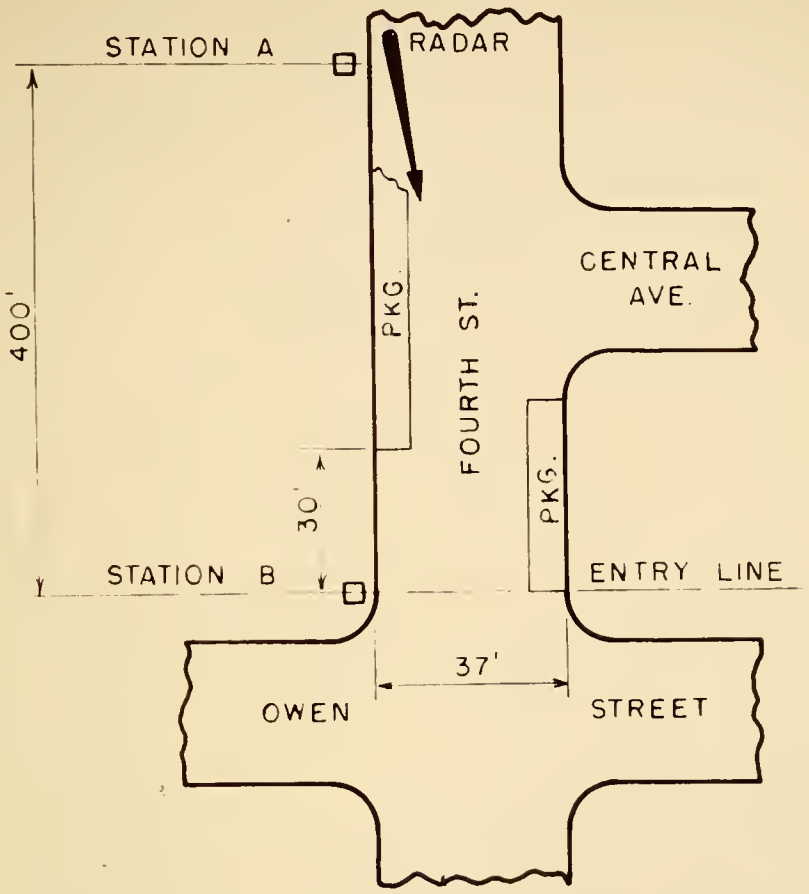


FIGURE 12. RELATION OF AVERAGE DELAY TO STANDARD LANE VOLUME AND RED PHASE LENGTH FOR 80 SECOND CYCLE.

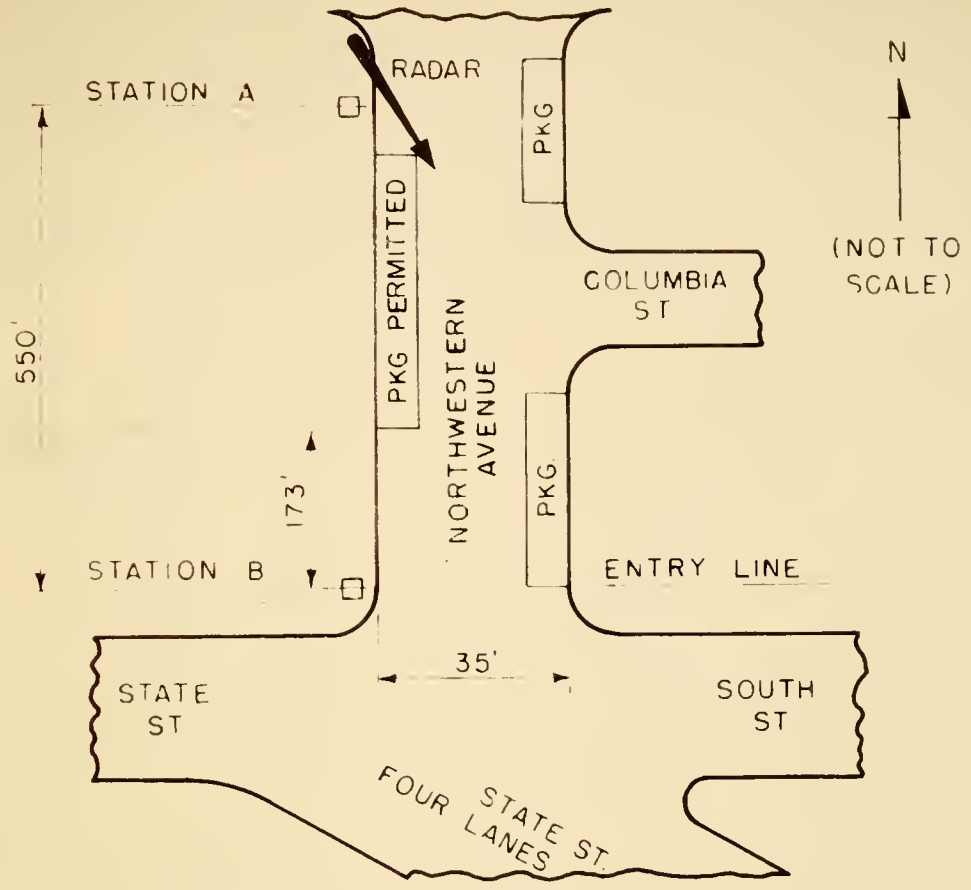


SIGNAL CHARACTERISTICS		60 SEC. CYCLE	65 SEC. CYCLE
57 %	APPROACH GREEN	34.2 SEC. G.	37.1 SEC. G.
6 %	" AMBER	3.6 SEC. A.	3.9 SEC. A.
37 %	" RED	22.2 SEC. R.	24.0 SEC. R.

CALCULATED CAPACITY = 1097 = 1100 VEH./GREEN HR.

BASED ON 4 % TRUCKS
8 % LEFT TURNS
7 % RIGHT TURNS

FIGURE 13. OBSERVED APPROACH CONDITIONS, FOURTH ST. AT OWEN ST., LAFAYETTE, INDIANA.



SIGNAL CHARACTERISTICS		60 SEC. CYCLE	70 SEC. CYCLE
30 % APPROACH	GREEN	18 SEC G	21.0 SEC G
5 % "	AMBER	3 SEC A.	3.5 SEC A
65 % "	RED	39 SEC R	45.5 SEC R

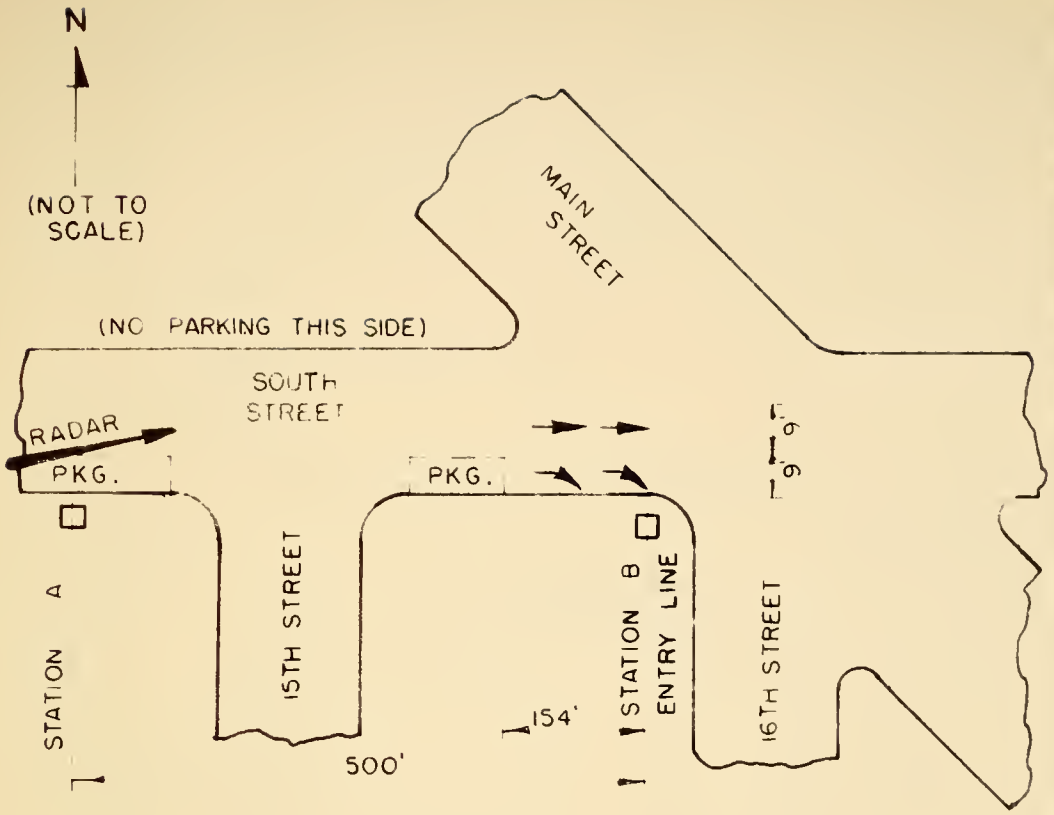
CALCULATED CAPACITY = 1248 = 1250 VEH./ GREEN HR

BASED ON 3 % TRUCKS

0 % LEFT TURNS

11 % RIGHT TURNS

FIGURE 14. OBSERVED APPROACH CONDITIONS, NORTHWESTERN AVE. AT STATE ST., W. LAFAYETTE, INDIANA.



SIGNAL CHARACTERISTICS			60 SEC. CYCLE	80 SEC. CYCLE
44 %	APPROACH	GREEN →	26.4 SEC. →	35.2 SEC →
25 %	"	GREEN	15. SEC. G.	20. SEC. G.
4 %	"	AMBER	2.4 SEC. A.	3.2 SEC. A.
71 %	"	RED	42.6 SEC. R.	56.8 SEC. R.

CALCULATED CAPACITY = 1915 VEH./GREEN HOUR

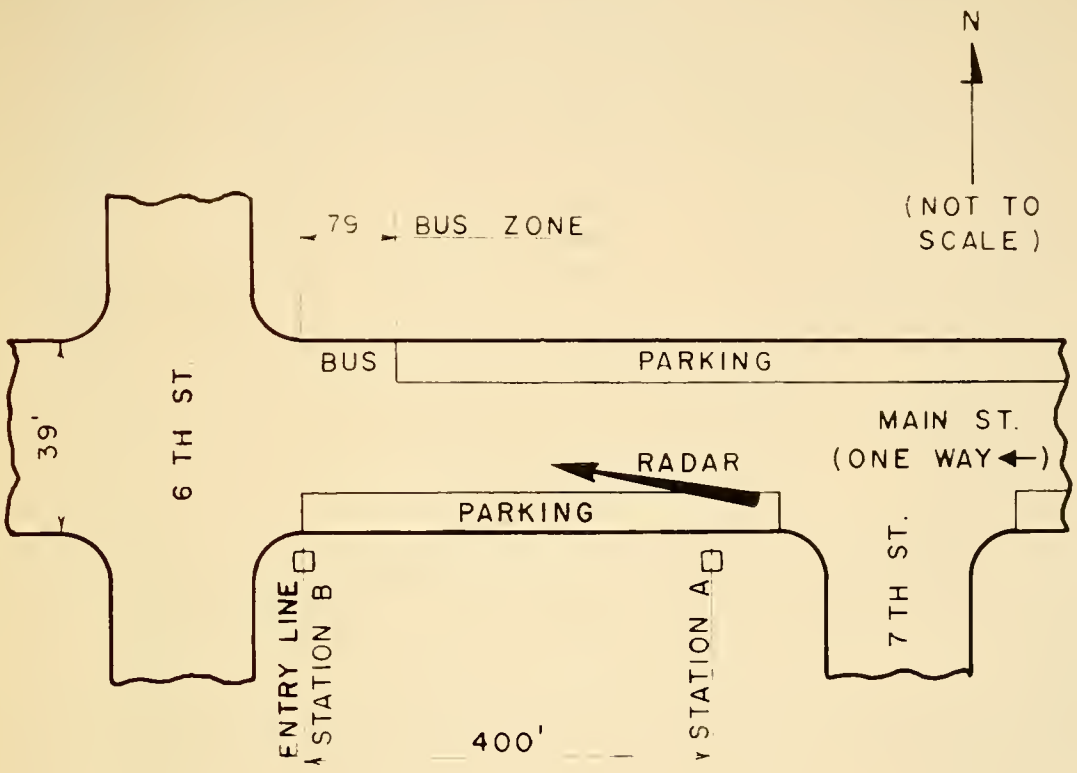
BASED ON 4 % TRUCKS

1 % LEFT TURNS

0 % RIGHT TURNS

AND ACTUAL RIGHT TURNING VOLUMES.

FIGURE 15. OBSERVED APPROACH CONDITIONS, SOUTH ST. AT MAIN ST., LAFAYETTE, INDIANA.



SIGNAL CHARACTERISTICS — 64 SECOND CYCLE (WORN CONTROLLER)

26	SECS. APPROACH	GREEN
3	SECS. "	AMBER
35	SECS. "	RED

CALCULATED CAPACITY = 1558 = 1560 VEH./GREEN HR.

BASED ON 3% TRUCKS
12% LEFT TURNS
11% RIGHT TURNS

FIGURE 16. OBSERVED APPROACH CONDITIONS, MAIN ST. AT SIXTH ST., LAFAYETTE, INDIANA.

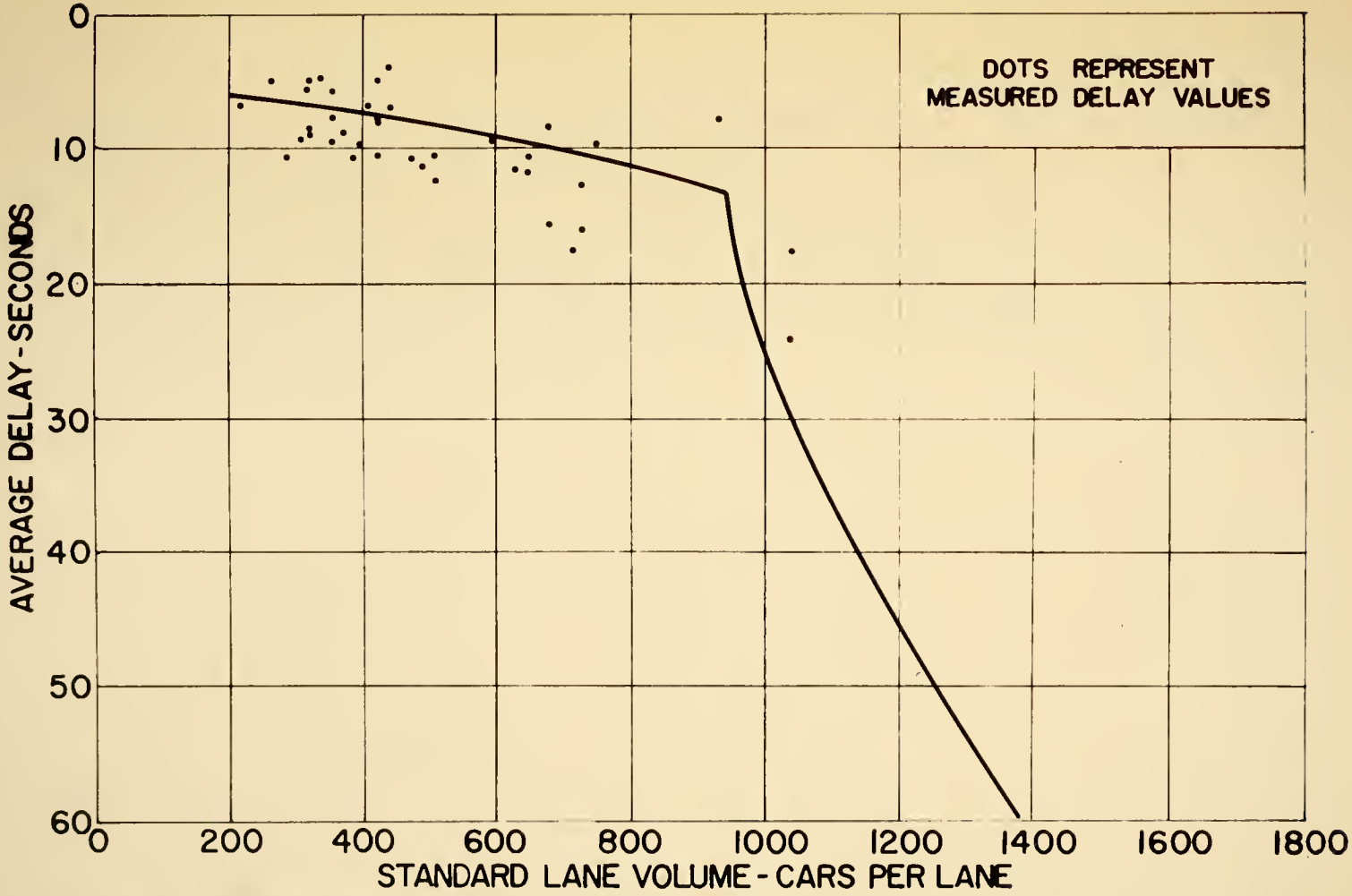


FIGURE 17. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, FOURTH AT OWEN, 60 SECOND CYCLE, 22 SECOND RED PHASE.

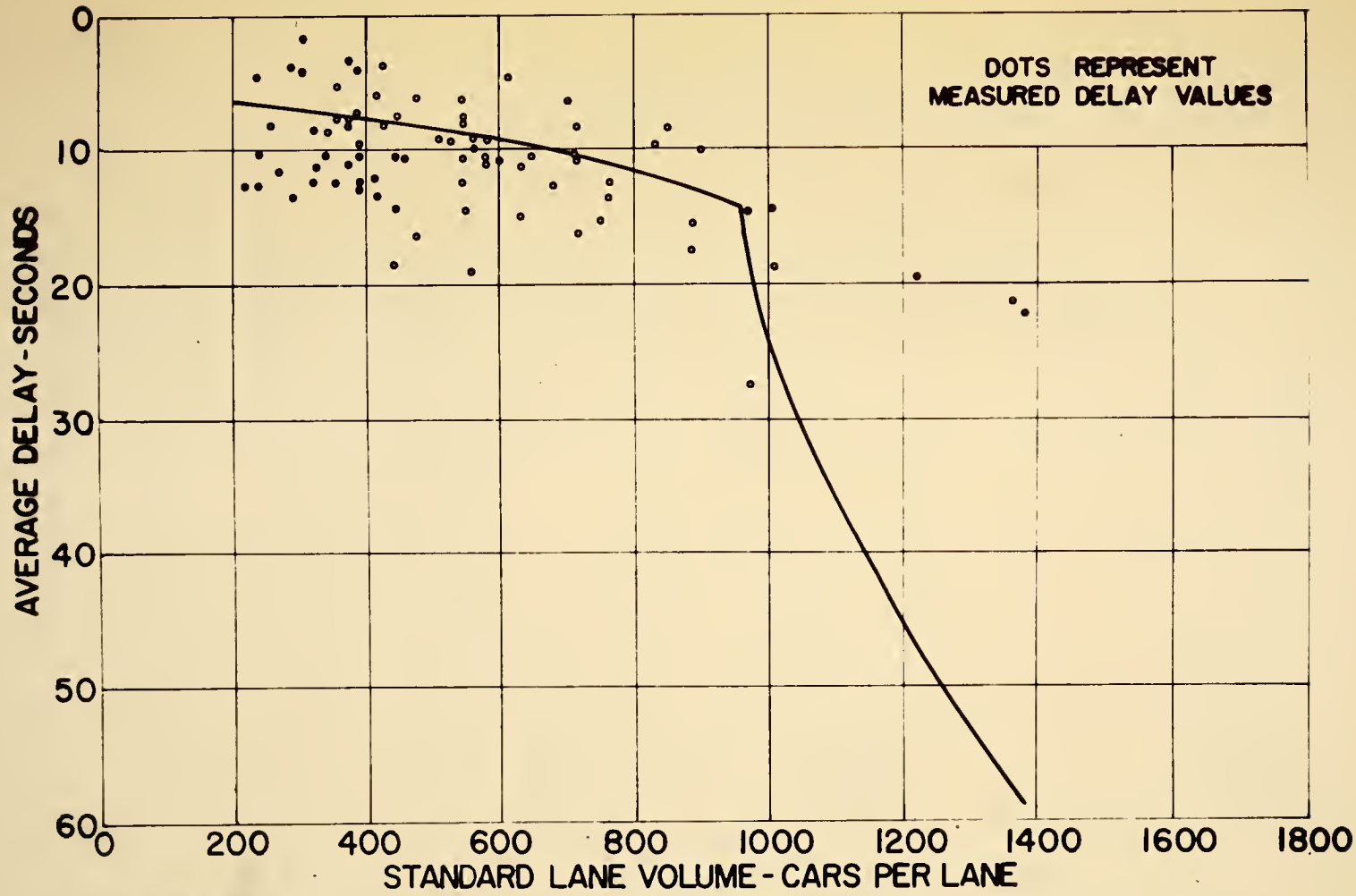


FIGURE 18. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, FOURTH AT OWEN, 65 SECOND CYCLE, 24 SECOND RED PHASE.

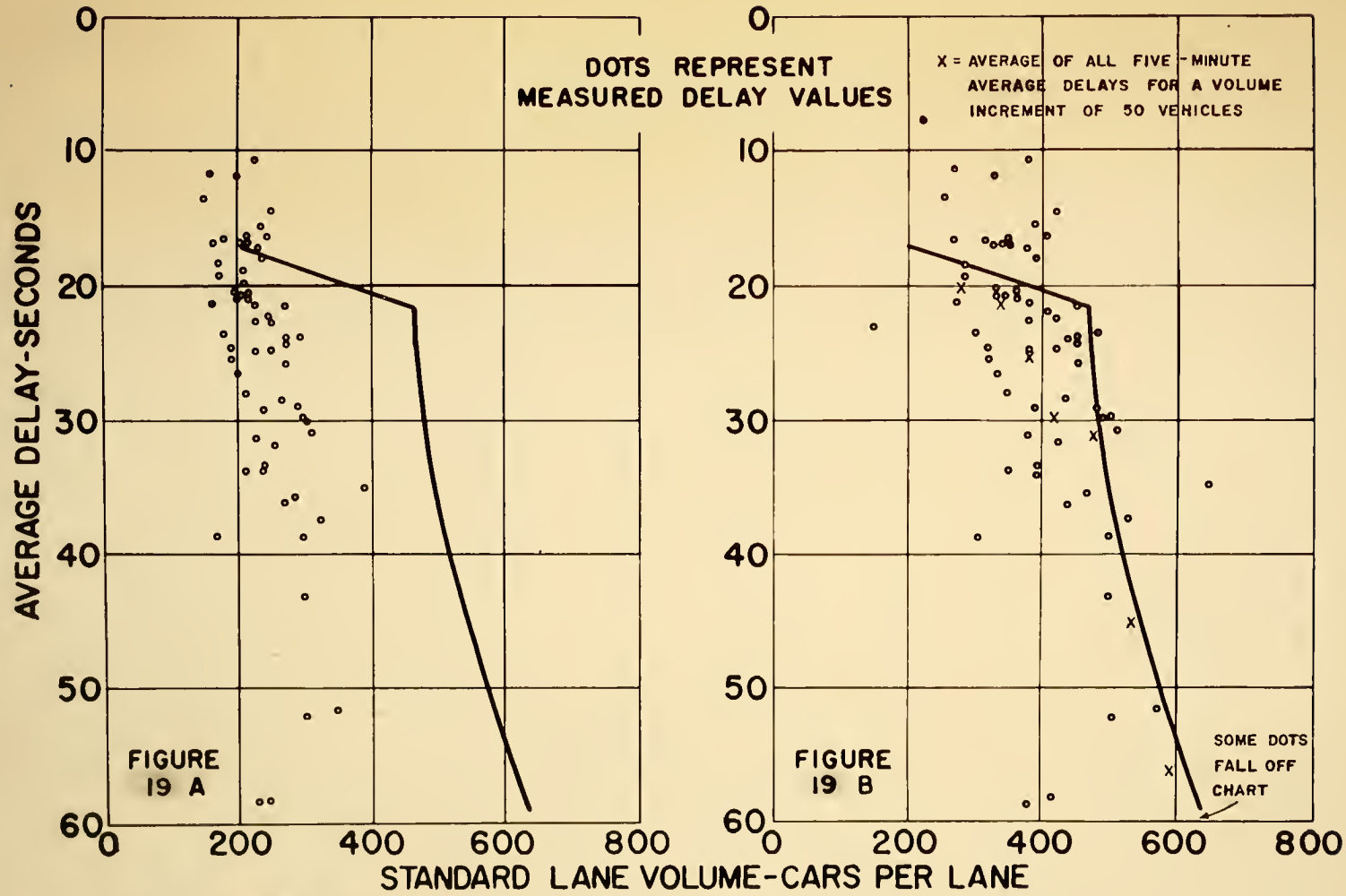


FIGURE 19. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, NORTHWESTERN AT STATE, 60 SECOND CYCLE, 39 SECOND RED PHASE. TWO CASES.

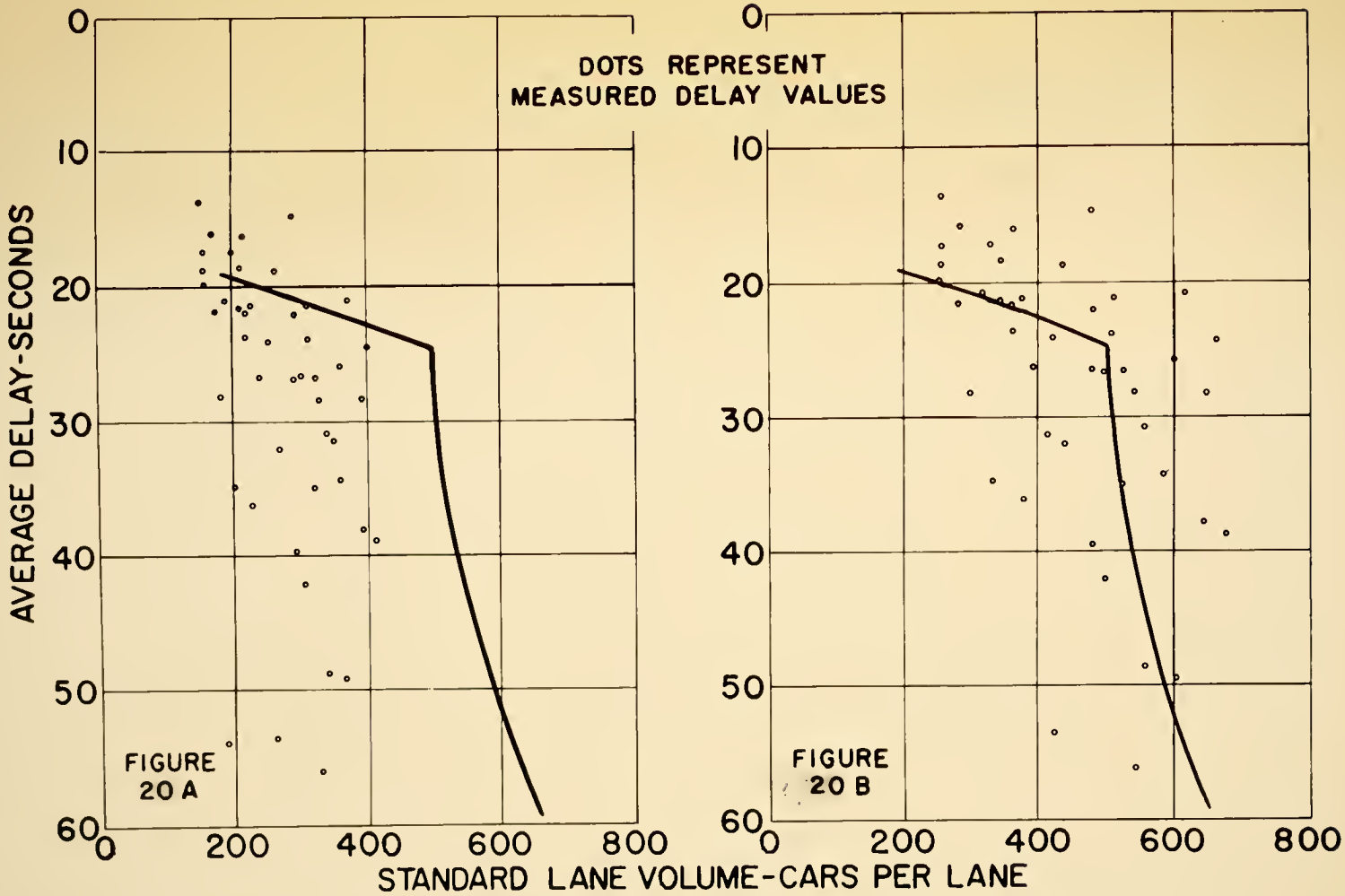


FIGURE 20. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, NORTHWESTERN AT STATE, 70 SECOND CYCLE, 45 SECOND RED PHASE. TWO CASES.

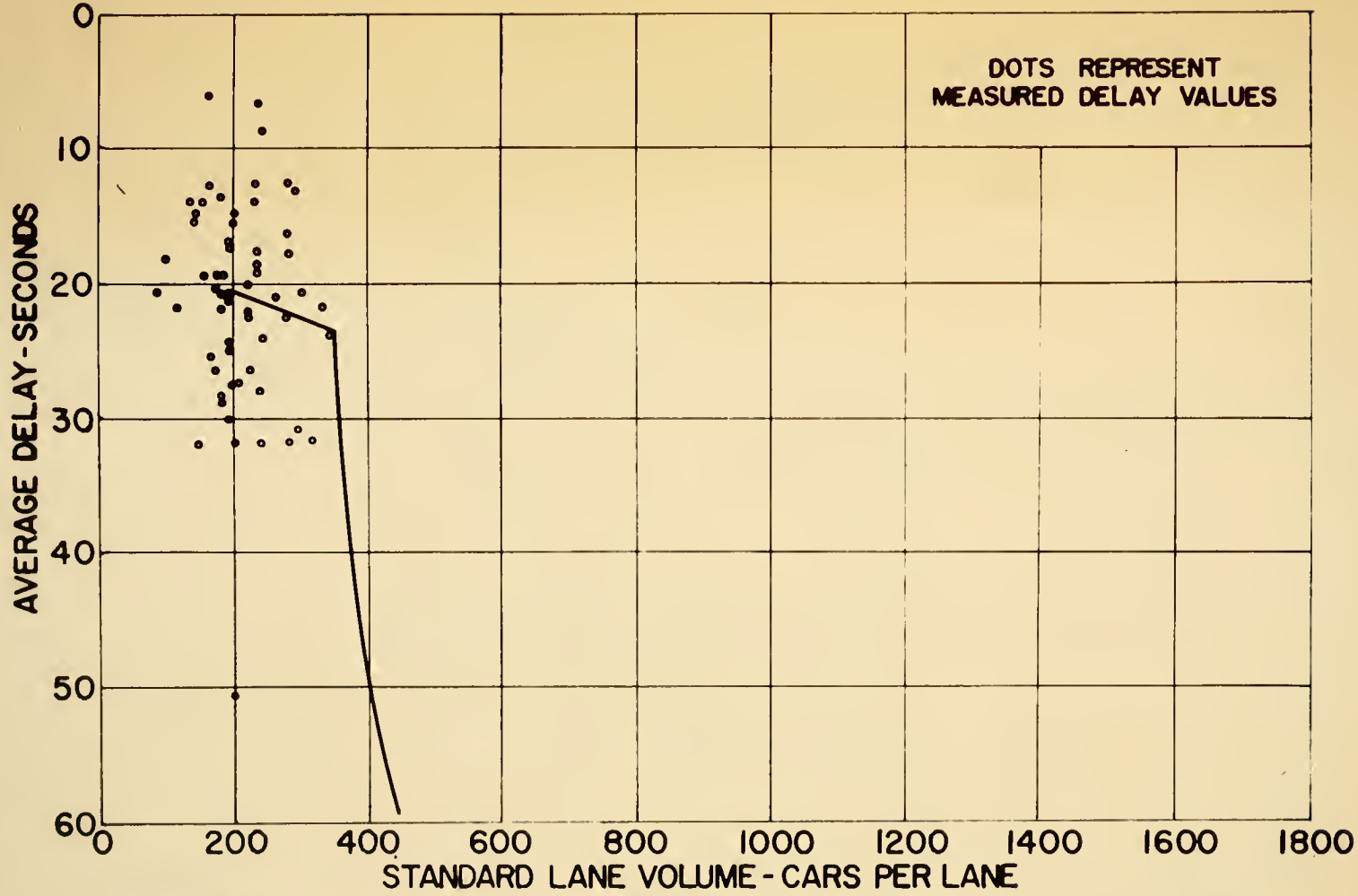


FIGURE 21. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, SOUTH AT MAIN, 60 SECOND CYCLE, 43 SECOND RED PHASE.

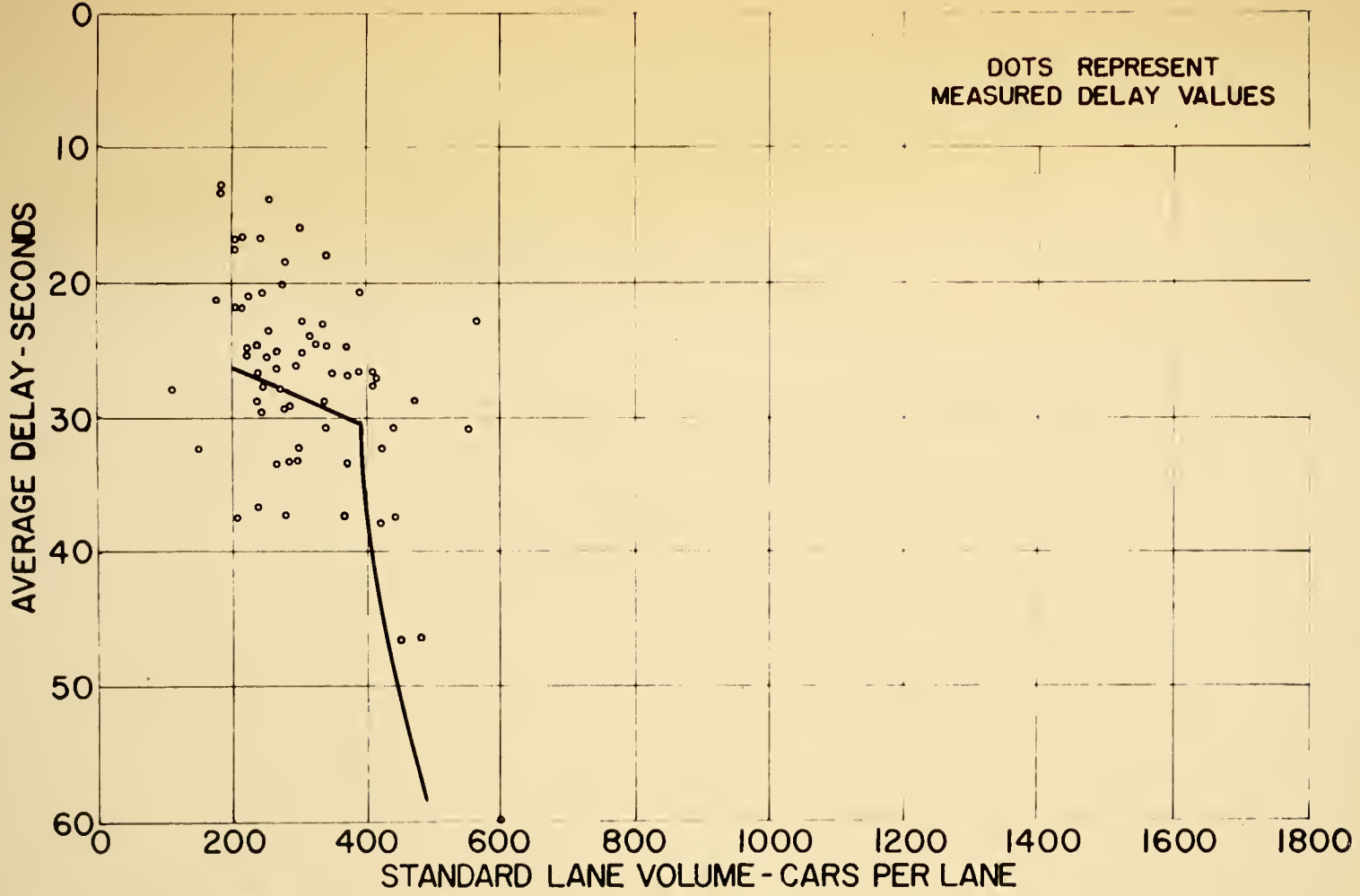


FIGURE 22. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, SOUTH AT MAIN, 80 SECOND CYCLE, 57 SECOND RED PHASE.

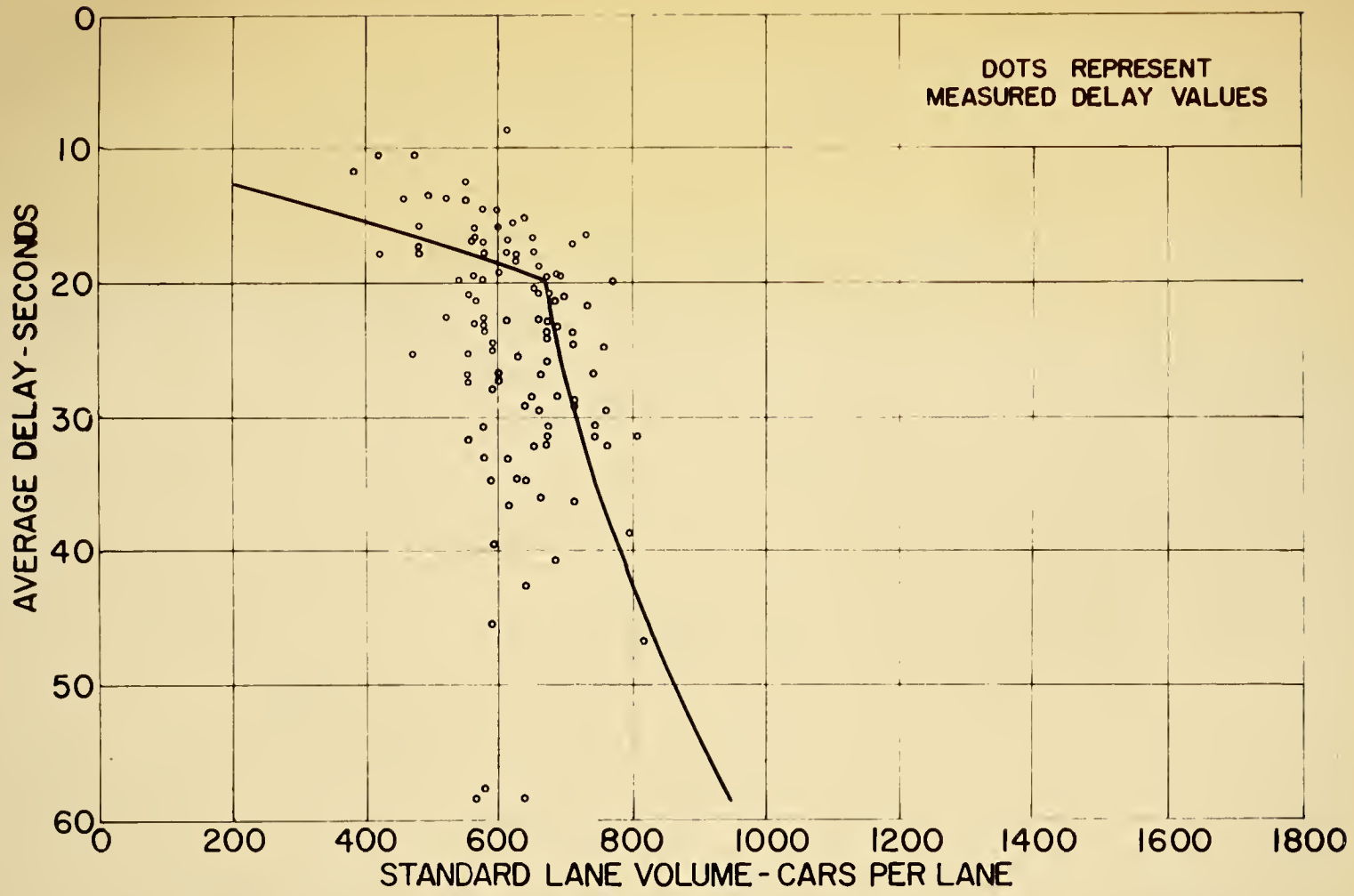


FIGURE 23. COMPARISON OF MEASURED AND THEORETICAL DELAYS WITH FIVE-MINUTE DATA GROUPING, MAIN AT SIXTH, 65 SECOND CYCLE, 35 SECOND RED PHASE.